# JOURNAL OF THE INSTITUTION OF CIVIL ENGINEERS.

No. 2. 1941–42.
DECEMBER 1941.


Paper No. 5227.

## "The Emerson Barrage."


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(*Ordered by the Council to be published with written discussion.*)[1]

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### Introduction.

The Emerson barrage is situated at Trimmu on the Chenab river, just below its confluence with the Jhelum, the site being about 14 miles south-west of Jhang, the headquarters of the district of the same name in the Punjab, India.

The purpose of the barrage is to bring under control the waters of the Chenab at this site, which vary in discharge from 900 to 650,000 cusecs, and to permit their use up to a maximum discharge of 7,750 cusecs for irrigation in the canals of the Haveli project.

The barrage is 3,026 feet long between abutments, and comprises a central weir section with an undersluice section at each end, separated from the weir by abutment piers 25 feet wide, each containing a fish ladder. The weir section has thirty-seven spans of 60 feet, with single gates 15·5 feet high. There are eight spans in the left undersluice, and six in the right, each of 30 feet, and all are equipped with double gates having a total height of 21 feet. All the piers are 7 feet thick. The difference in level

[1] Correspondence on this Paper can be accepted until the 15th April 1942, and will be published in the Institution Journal for October 1942.

between the top of the gates and the cistern floor is 29 feet (the maximum head which the work is designed to withstand) in the case of the undersluices, and 25 feet in the case of the weir. The undersluice pockets are separated from the weir by divide walls, 360 feet long on the upstream side and 405 feet long downstream. The left pocket is further divided centrally upstream by a wall 560 feet long, and the section between the wall and the regulator is provided with a silt-excluder slab 231 feet long covering the whole width of the pocket, and supported on piers 37feet apart centre to centre. The right pocket has a small excluder slab covering only one bay of the undersluices. From the left pocket the Haveli canal takes off through a regulator comprising five spans of 24 feet with gates 8 feet high, operating behind a reinforced-concrete breast-wall. On the right, the Rangpur canal regulator has three spans of 24 feet with gates 5·5 feet high. All of the gates are fully counterbalanced, and are fitted with standing gearing for hand operation only, and with anti-friction rollers of the live Stoney type for the barrage, and of the large-diameter fixed type for the regulators.

The barrage is linked to high land on both sides by marginal bunds, 9 miles long on the left and 11 miles long on the right. Armoured guide banks of Bell's pattern are provided, extending 3,500 feet upstream and 800 feet and 600 feet downstream on the right and left respectively.

*The Haveli Project.*

Inundation canals are those which can flow only when the annual rise in the river, during the monsoon, produces water-levels high enough to command them. This occurs for only 3 or 4 months between June and September each year. Canals, the command of which is assured throughout the year by permanent diversion works on the rivers, are designated perennial, non-perennial, or semi-perennial: the first being those canals to which the limited winter supplies of the river are allotted and which consequently run throughout the year, whilst the second are alloted water from April to October only: non-perennial supplies give ample water for the summer (*Kharif*) crops and provide water for the sowing of the winter (*Rabi*) crops which can be matured with the help of winter rains and well-irrigation. Semi-perennial canals are those which are permitted to flow in the winter, but for which the winter supplies are usually inadequate.

The Haveli project (Fig. 1, Plate 1) will provide a perennial supply for the area which formerly received a semi-perennial supply from the Sidhnai canals commanded by the Sidhnai headworks on the Ravi river. It will convert to non-perennial conditions the areas served by the inundation canals of the Multan district on the left bank of the Chenab and the Muzaffargarh district on the right bank. It will also irrigate all areas at present not receiving irrigation which are commanded by its various canals. Part of this land is Government property, which will be sold to help defray the cost of the project. Where such land can be irrigated

independently or combined with old perennial areas it is given a perennial supply. New proprietary land is given a non-perennial supply, except when it can be irrigated only from perennial channels.

The areas affected directly by the project are as follows :—

| | | |
|---|---|---|
| Old area to be irrigated perennially | 547,000 | acres. |
| „ „ „ non-perennially | 369,000 | „ |
| New perennial area : Crown waste | 155,000 | „ |
| „ „ „ proprietary | 25,000 | „ |
| New non-perennial area : Crown waste | 51,000 | „ |
| „ „ „ proprietary | 325,000 | „ |
| Total | 1,472,000 | acres. |

The project will provide these areas with water at rates of 3·0 and 4·8 cusecs per 1,000 acres for perennial and non-perennial areas respectively at distributary outlets, that is, the points where the water is taken over by the cultivators.

The Sidhnai canal having been semi-perennial, there were occasions when water was surplus to the requirements of the Upper Bari Doab canal, which, offtaking from Madhopur in the upper reaches of the Ravi river, has first claim on its water. Formerly this water was utilized at Sidhnai, but with Sidhnai requirements being met from the Chenab, it can now be utilized in other canals. Some of this water will enable an area of 142,000 acres of the Lower Chenab canal, known as the Burala branch extension, formerly semi-perennial, to be made perennial. To effect this, the water will actually be used in the Lower Bari Doab canal from the Balloki headworks on the Ravi, replacing water brought from the Marala headworks of the Upper Chenab canal on the Chenab, which will be diverted by way of the Chenab river to the Khanki headworks of the Lower Chenab canal on the same river. The balance of the surplus Ravi water will be used to supplement the supply of the Pakpattan canal, dependent on the Sutlej river, through a 700-cusec link joining the Lower Bari Doab canal to this channel, where it is expected to irrigate 120,000 acres annually.

Apart from the Emerson barrage and the Pakpattan link the Haveli project comprises :—

(*a*) The Rangpur canal. This is situated on the right bank of the Chenab, has a capacity of 2,710 cusecs, and is 89 miles long up to the point where it tails into the existing Muzaffargarh inundation canals. It traverses a narrow strip of good land lying between the river and the Thal desert, and is of special interest in that along certain reaches, totalling 8 miles in length, the desert is so close to the river that it is necessary to align the canal through the desert. This involves cutting through sandhills to depths of as much as 40 feet.

(*b*) The Haveli canal. This channel, which is 45 miles long and has a capacity of 5,200 cusecs with a slope of 1 in 10,500, is the main feeder which transmits the Chenab water from Trimmu to Sidhnai. It crosses

the Shorkot Road—Khanewal section of the North Western railway in the vicinity of the Shorkot Road station, and falls into the Ravi just above the Abdul Hakim railway-bridge 5 miles from Sidhnai. This canal is chiefly of interest because it is lined throughout, the lining consisting of two courses of 2½-inch brick tiles, enclosing a course of 1 : 3 cement plaster ½ inch thick. A mesh of ¼-inch bars, 12 inches by 12 inches on the sides and 24 inches by 24 inches on the bed, is provided, being placed at the lower surface of the plaster, the upper surface of the lower course of tiles being grooved to receive it. The waterway is generally 12 feet deep, the bed-width varying from 71 to 85 feet with 1-to-1 side-slopes joined to the bed with a 15-foot radius curve.

(*c*) Raising and strengthening the Sidhnai dam. This work consisted of thirty-two spans of 20 feet each, with a depth of waterway of 9 feet, equipped for needle regulation. It has been raised by 2·5 feet, strengthened, and provided with a cut-off line of sheet-piling. A public road-bridge across the dam is also being embodied in the remodelling of this work.

(*d*) The project also includes the enlargement of the head reach of the Sidhnai canal, the linking of this canal with the left bank inundation canals, provision of distributaries for the new areas, and adaptation of the old distributaries to the new conditions of supply.

The total cost of the project is likely to be about 35,000,000 *rupees*, of which 15,000,000 *rupees* represents the cost of the barrage.

## Theory of the Design of Weirs on Sand.

The Emerson barrage is the first great barrage to be constructed in the Punjab in which full advantage has been taken of the advance in the theory of the design of weirs on sand foundations. A complete treatment of the theory of the subject is given in a publication on weir-design on permeable foundations[1].

The essential purpose of a barrage of this type is to head up low river supplies in order to command the canals taking off from the river above it. The work is therefore liable to a static hydraulic head which, since the site is an alluvial river-bed, causes seepage-flow through the pervious material under the foundations. Subsoil flow threatens the stability of the work in two ways: (*a*) it sets up a pressure-gradient in the river-bed at the toe of the work which, if excessive, may cause movement of the bed, with consequent failure of the work. This is known as "piping"; (*b*) pressures are created under the work itself which, if not adequately contained, will burst through it, providing a short path for subsoil flow and leading to total failure of the work by "piping."

[1] A. N. Khosla, N. K. Bose, and E. McKenzie Taylor, "Design of Weirs on Permeable Foundations." Central Board of Irrigation, India. Publication No. 12. 1st September 1936.

The design of the barrage has to provide for the escape of surplus water over the work and has to be safe against the dynamic effects of such flow. These effects may be divided into two classes : (*c*) the escape will usually be accompanied by a sudden drop in level as the water passes over the work. The energy thus set free produces heavy turbulence, against the action of which the downstream bed must be protected. (*d*) A given discharge per unit width requires a certain depth for bed equilibrium depending upon the material of which the bed is composed. The maximum depth required for this purpose must be estimated upstream and downstream of the work, and suitable protection must be provided for any portion of the bed where this depth cannot safely be allowed to develop. If the protection required for (*c*) and (*d*) is inadequate, the effect of the resulting scour will be to shorten the path of seepage-flow under the work, and in this case also ultimate failure will be caused by " piping."

At the time of the design of the Sutlej Valley weirs, whilst all these phenomena were known, they were not clearly understood. The criterion of safety against " piping " was the ratio of the length of flow, measured along the under-surface of the work, to the hydrostatic head ; a value of 18 being taken as safe for a fairly fine grade of sand. This led to the floors being made excessively long, and safety was secured only because a cut-off of some kind was invariably provided at the downstream end to resist dynamic action. The seepage-pressure under the work was calculated on the assumption that loss of head was directly proportional to the length of flow ; cut-offs, of masonry or sheet-piles, were rightly considered of high value, but only because both sides could be counted on for increasing the length of flow. The pressure-gradient obtained by this method was, however, roughly similar to that which occurred in practice. The phenomenon of the standing wave was not generally understood, with the result that in practice it was allowed to occur where protection was inadequate, and in the case of the Islam weir caused a major failure.

The major improvements which have taken place since Sutlej Valley days have undoubtedly been :—

(*a*) Recognition of the fact that seepage flow under a weir is of a streamline nature, conforming to Darcy's law $q = Ka.\frac{dh}{dl}$, where $q$ denotes the discharge at any point of a stream-tube, $K$ is a constant for any material, known as its transmission-constant, $a$ denotes the cross-sectional area of the tube, and $\frac{dh}{dl}$ denotes the pressure-gradient at the point.

The transmission-constant is a function of the viscosity of water, and hence varies directly with temperature. For a given weir profile and uniform transmission-constant there can be only one conformation of streamlines and contours of equal pressure in the vicinity of the work, and this conformation can be accurately determined mathematically for

simple cases. Close approximation can also be obtained mathematically and experimentally for complicated cases.

The simplest way of determining the flow-net experimentally is to make use of the analogy between Darcy's law and Ohm's law. A model is built up in a tray containing an electrolyte of uniform thickness, in which the boundaries of flow are represented by non-conductors, and the surfaces of inflow and outflow by conductors. By applying a voltage (representing the hydraulic head) to the conductors, an electric current (analogous to the discharge) is caused to pass through the electrolyte, and by observing the potential at various points and plotting the results, the contours of equal potential, representing the contours of equal pressure in the prototype, can be obtained. Streamlines will cut these contours orthogonally, and can be plotted accordingly, or alternatively they may be obtained directly by substituting conductors for non-conductors on the flow boundaries and non-conductors for conductors on the surfaces of inflow and outflow, when the equipotential contours will represent the streamlines. Given the flow-net, the pressure-gradient and pressure at any point under a weir can be determined with accuracy.

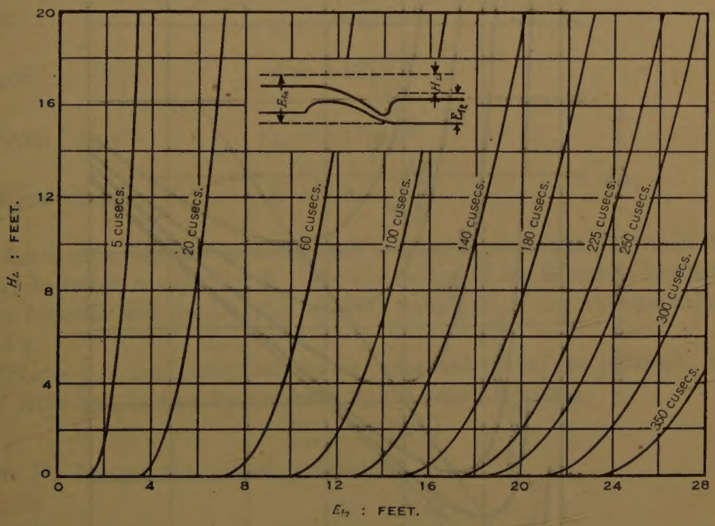
The theory, of course, assumes uniformity in the sub-foundation, of both transmission-constant and temperature. Field observations have proved, however, that when the river-bed is reasonably uniform, namely, when there are no extensive clay strata, the theoretical conformation is reasonably accurate. It is interesting to note, however, that variations due to temperature have actually been observed in the field. The passage of the water through the subsoil is, of course, very slow; in fact, several months are required for water to pass under a large weir. The water, however, largely retains the temperature at which it enters the subsoil and consequently, it happens that at times the water at the lower end of a work (representing water which entered the subsoil during the summer) is warmer than that at the upper end, which entered several months later in the winter, and vice versa. The result of this temperature-difference is a relatively higher transmission-constant at the lower end, and a general reduction of pressure under the work, or vice versa.

A safety factor of about 15 per cent. is, however, sufficient to cover such pressure-differences. In any case, the modern barrage is equipped with an extensive system of pressure-pipes by means of which the pressure at all important points can be kept under observation. It is rarely required to operate at the maximum designed head, and it would not be called upon to do so if the observed pressures indicated that to be unsafe.

(*b*) The next point of importance in which the modern theory of weir design has advanced is a better understanding of the phenomenon of "piping." It is now recognized that failure from this cause can occur only at the surface at which the subsoil flow emerges into the river bed and that it will not occur if the weight of the sand is not overcome by the pressure-gradient of subsoil flow. Consideration of vertically-upward flow

in a stream-tube of unit cross-sectional area will show that failure will occur when the gradient $\frac{dh}{dl}$ at exit is $(\rho-1)$, where $\rho$ denotes the specific gravity of the material. As the value of $\rho$ is never very far from 2·0 it may be taken, for all practical purposes, that failure will occur when the exit gradient is 1·0. It may be argued that the exit gradient is not necessarily the highest gradient, and that the mean gradient over a length of stream-tube terminating in the surface of outflow may be greater than the slope at the surface itself. This is, in fact, true in the common case of a vertical cut-off at the toe of a weir, but in such cases the highest gradients occur at the flow-boundary, namely, the under-surface of the weir, and

*Fig. 2.*

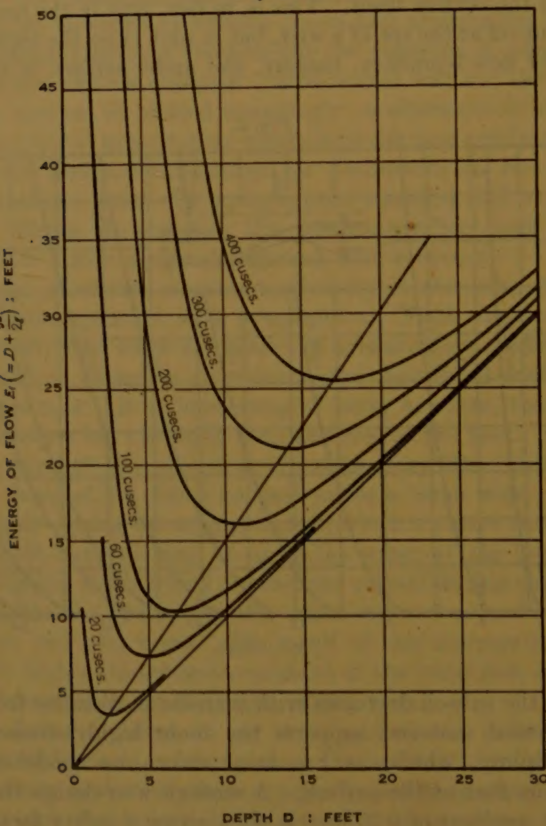



the stress in the subsoil decreases with increase in distance from the weir. This low-stressed material supports the more highly-stressed material, preventing failure, which, as has been shown by model-experiments, actually occurs first at the surface. A modern weir design therefore provides an exit gradient of 0·25 or 0·2, thus giving a safety factor of 4 or 5 against failure by "piping."

(*c*) The third point is the recognition of the use of the standing wave in localizing the dissipation of the energy liberated by a fall of water. The modern weir is provided with a cistern at a level low enough to ensure that any hypercritical flow set up by the weir cannot pass beyond it, and is of sufficient length to permit the excessive turbulence of the wave to abate before the water leaves it. In older works it frequently happened that the turbulence took place where the bed was protected only by pervious pro-

tection consisting of concrete blocks or loose stone. Such protection is not suitable for the violent action of a wave, which may erode the underlying bed through its interstices. Consequently the modern cistern is embodied in the impervious portion of the work. The mathematics of the standing wave are now so well known that they need not be repeated. The forms of two of the series of curves used in the design of the barrage are, however, illustrated. The first series (*Fig. 2*), indicates in a convenient form the

*Fig. 3.*




relation between the discharge per foot run, afflux, and cistern-depth, whils the second (*Fig. 3*) indicates, for any discharge per foot run and tot energy-depth, the depths of subcritical and hypercritical flow. Fro these curves the level of the cistern floor can be determined exactl There is, however, no precise method of determining the requisite lengt of the cistern. Experience shows that most of the turbulence has subside in a length equal to five times the height of the wave, and this should b provided as a minimum.

During recent years considerable research has been carried out by means of model-experiments on the subject of reducing action below falls by varying cistern dimensions and shapes, and by the introduction of baffle-walls and blocks on the floor, which have the effect of reducing bed-velocities. From the results of many such experiments in the Punjab Irrigation Research Institute, the conclusion has been reached that the simplest device which will produce the desired result consists of rows of staggered blocks, the size and position of which is best determined experimentally.

(*d*) The depth of non-scouring flow may be obtained from Kennedy's or Lacey's critical-velocity formulas.

From Kennedy, if $F$ denote the critical velocity ratio of the bed silt,

$$V_0 = 0{\cdot}84Fd^{0{\cdot}64}$$

$$q = 0{\cdot}84Fd^{1{\cdot}64}$$

whence $$d = 1{\cdot}12\left(\frac{q}{F}\right)^{0{\cdot}61}$$

From Lacey's $$V_0 = 1{\cdot}17\sqrt{fR}$$

it follows that $$R = 0{\cdot}9 \times \sqrt[3]{\frac{q^2}{f}}.$$

and for a wide, shallow section, such as that of an alluvial river in flood, $d$ may be taken as equal to $R$. For the design of the Emerson barrage the latter formula was used. *Fig. 4* gives values of $R$ for various values of $f$ and $q$.

Where $q$ is not controlled by an adjacent work, for instance, in an open river-bed, it may be obtained from Lacey's formula:

$$P_w = 2{\cdot}67\sqrt{Q}$$

where $P_w$ denotes the wetted perimeter;

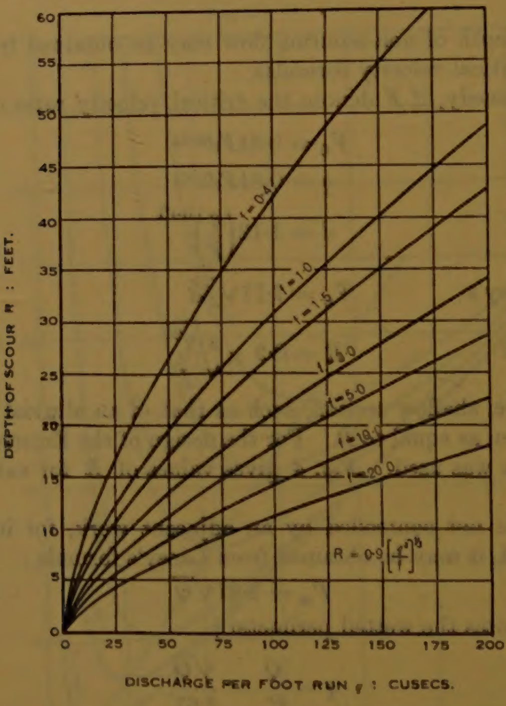
therefore $$q = \frac{Q}{P_w} = \frac{\sqrt{Q}}{2{\cdot}67}.$$

In design, a safety factor ranging from 1·26 to 2·25 should be applied to the normal scour-depth, in accordance with the river action likely to occur at the location under consideration: a value of 1·25 would apply to gently-guided flow, such as that along the shanks of the guide-banks; 1·50 would apply to the approach to the weir; 2·0 would apply to the bed downstream of a weir where the wave turbulence persists in a modified form for some distance; and 2·25 would be suitable for the nose of guide-banks or spurs. Between the nose and the shanks of such works the factor would obviously be varied between the limits of 2·25 and 1·25.

The mathematical determination of the flow-net will now be further considered. Prior to 1932 mathematical solutions existed for only two simple cases, namely a plain floor, resting on, but not penetrating, the

subsoil; and a simple pile-line with the bed at the same level upstrea
and downstream. In these cases the streamlines are the ellipses, and th
pressure contours are the hyperbolas, of conics confocal to the ends, i
the case of the floors, and to the bottom end and a point elevated abov
the bed to a height equal to its depth, in the case of the pile. In 193
Weaver[1], using Schwartz Christoffel transformations of the confocal conic

*Fig. 4.*




to deformed axes of co-ordinates, obtained a solution of the case of a simpl
horizontal floor combined with a vertical pile-line at any point in its lengt
This, of course, includes the valuable particular case of a floor with a pil
line at the end. Using the same method, Bose and Malhotra, of the Irr
gation Research Institute, Lahore, have since obtained solutions of th
following cases: (i) a floor with a pile-line at the downstream end, an
the downstream bed at levels varying from that of the floor. (ii) a pil
line with the bed at different levels upstream and downstream. (iii) a
inclined floor. (iv) a depressed floor.

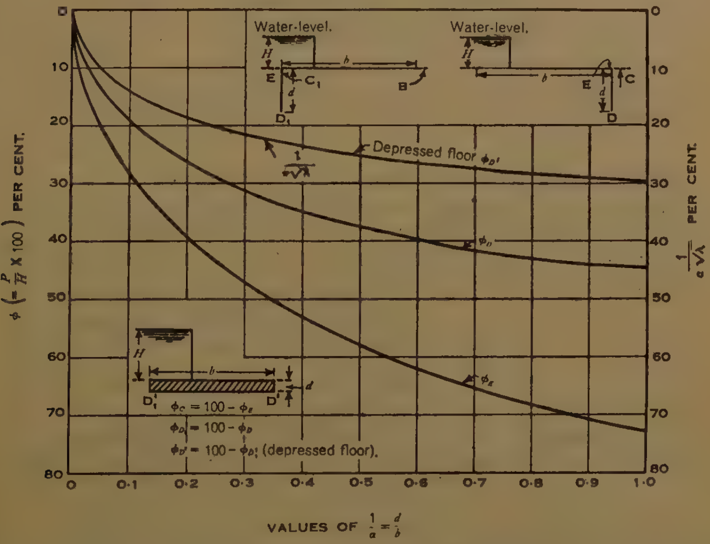
[1] Warren Weaver, "Uplift Pressure on Dams." Journal of Mathematics a
Physics, vol. xi (1932). (No. 2, June, 1932.) Massachusetts Institute of Tec
nology Press.

From a study of the solution of these cases [1] and of model-experiments of more complicated cases, the following points may be observed:

(*a*) The exit gradient at the end of an undepressed horizontal floor situated at the end of a work is infinite. Such a work is therefore unsafe and should be avoided. In other words, all works must terminate in a vertical cut-off. The efficiency of a cut-off in reducing the exit gradient increases rapidly with its depth; hence the downstream cut-off should be as deep, relatively to the length of the floor, as is practicable.

(*b*) The pressure contours concentrate around salients in the profile, particularly the lower ends of cut-offs. Consequently there is a heavy loss

*Fig. 5.*




of pressure in the vicinity of such points. Conversely, the gradient along the floor between pile-lines is relatively regular, and for all practical purpose can be represented by a straight line.

(*c*) If cut-offs are placed close together, there is mutual interference and they lose their effectiveness. If they are not close together the pressure-distribution in the vicinity of any cut-off will be affected only very slightly by the presence of the others.

In other words, the pressure can be calculated for the known case of the combination of the floor and single cut-off, minor adjustments being made to compensate for the variations from standard conditions.

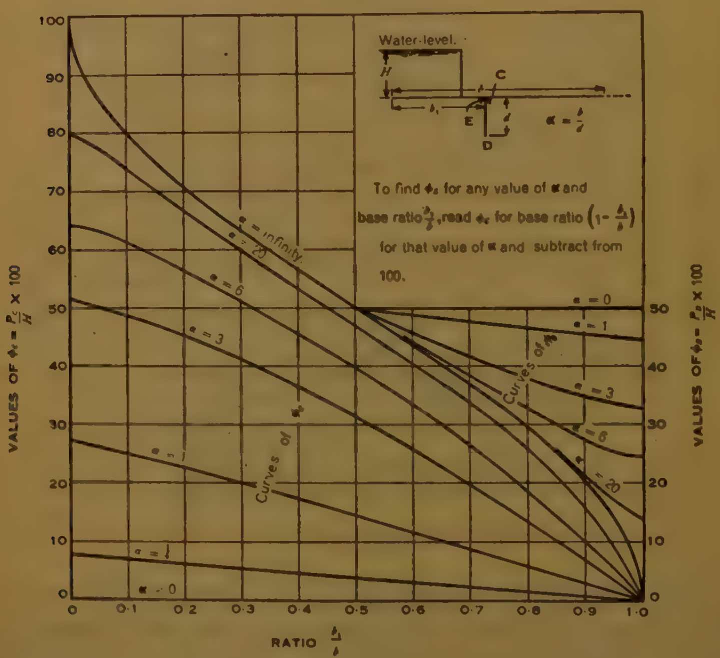
For use in design, the only information required is the pressure at the bottom of the pile-lines and at the junctions of the pile-lines and the

[1] A. N. Khosla, N. K. Bose and E. McKenzie Taylor, *loc. cit.*

floor, together with the exit gradients. This information is contained in curves obtained from Weaver's solution of the case of a floor and pile; some of the curves are reproduced in *Figs. 5* and *6*.

Following Khosla's notation, the symbol $\phi$ is used in this Paper to denote the proportion of the total head $H$ across the work which occurs at any point; that is, if $P$ denote the pressure at the point $\phi = \frac{100P}{H}$. The suffix $D$ applies to the bottom of the pile and $E$ and $C$ apply to the junction with the floor on the upstream and downstream sides respectively;

*Fig. 6.*




$\alpha$ denotes the ratio of the length of floor $b$ to the depth of pile $d$, and $b_1$ denotes the distance of the pile from the upstream end of the floor.

Variations in practice from the theoretical case to which the curves strictly apply, and the corrections which may be applied, are as follows:—

(*a*) A difference between the level of the underside of the floor in various sections, and between the floor and the bed upstream and downstream.

For the exit gradient and $\phi_D$ for the outer piles, $d$ is taken as the depth

[1] A. N. Khosla, N. K. Bose, and E. McKenzie Taylor, *loc. cit.*

below the river-bed. Variations in floor-level will have no appreciable effect upon these values.

For $\phi_C$ for the upstream pile, $\phi_E$ for the downstream pile and $\phi_D$ for intermediate piles $d$ is taken as the average depth on the two faces. For $\phi_E$ and $\phi_C$ for intermediate piles the actual depth of the corresponding face is taken.

(*b*) The effect of an adjacent pile.

Khosla gives the following formula :—

$$C = 19\sqrt{\frac{D}{b}\frac{(d-B)}{b}},$$

where $C$ denotes the percentage correction to be added to or subtracted from $\phi_E$, $B$ the distance between the piles, and $D$ the depth of the bottom of the pile causing the interference below the top of the pile under consideration. It is found that $\phi_D$ is unaffected by the pressure of adjacent piles, and that if there is more than one other pile in the work, only the first pile on the same side as the point at which the pressure is required need be considered.

This formula breaks down in the case of the effect of an outer pile on an inner pile if the latter is smaller than the former, and at a distance less than twice the depth of the outer pile. This case, however, is not likely to occur in practice.

(*c*) The effect of a sloping floor.

Khosla gives the following formula :—

$$C = \frac{b_2}{b_1} \cdot x$$

where $C$ is the correction to be added to $\phi_E$ or subtracted from $\phi_C$ if the floor slopes upwards from the pile. If the slope is downward, the correction is negative. $b_1$ denotes the distance to the next pile or the end of the floor, $b_2$ the length of the horizontal projection of the sloping portion of the floor, and $x$ the maximum difference between the theoretical values of $\phi$ at any point along a horizontal floor, and a floor with the given slope, the bed upstream and downstream remaining horizontal.

*Figs.* 7 indicate the pressure-distribution along a horizontal and an inclined floor, and also values of $x$ for different slopes.

The corrections given above are frankly empirical. They are, however, based upon a large number of theoretical and experimental results, and yield values which are accurate to within 2 or 3 per cent.

## Design of the Barrage.

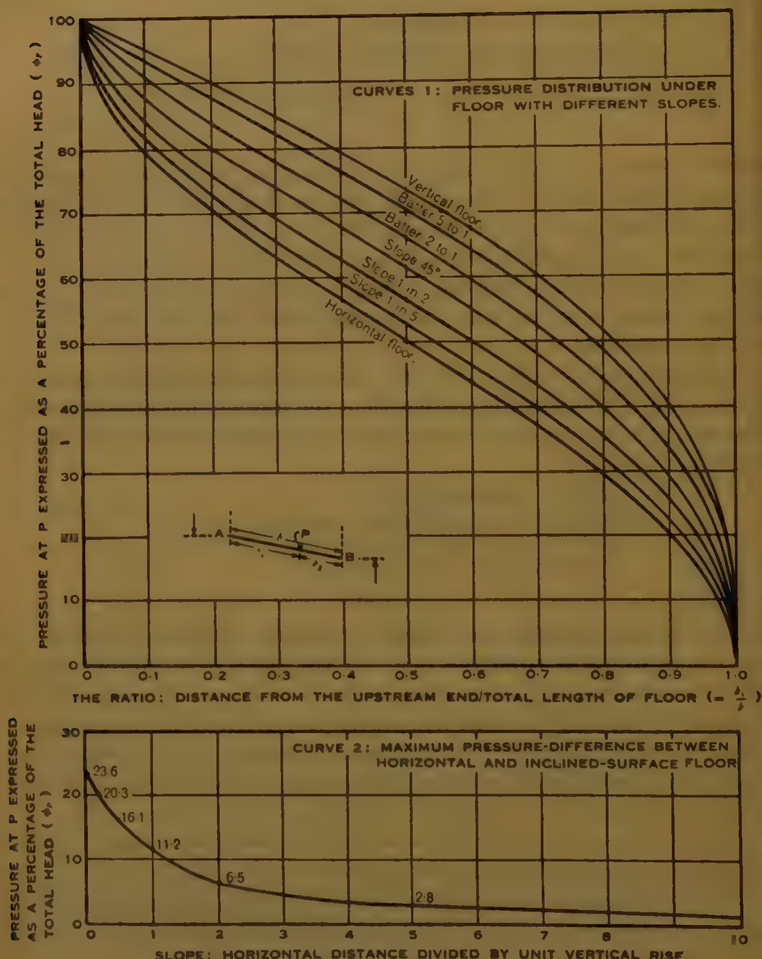
*River Discharges and Levels and Barrage Waterway.*

The framers of the Haveli project were fortunate in that the highest flood on record for the Trimmu site occurred in September 1929, while the

project was under preparation, and the officer in charge of the work was able to make a very detailed investigation of the discharge and levels which prevailed.

The figure actually used in the design of the barrage was 645,000 cusecs,

*Figs. 7.*




FLOOR PRESSURE-DISTRIBUTION CURVES.

which was that officially accepted after the 1929 investigations. The corresponding level, based on that occurring at that time at Trimmu and corrected for river slope between the gauge-site and the barrage-site, was 490·5.

The lowest discharge ever recorded at Trimmu was 806 cusecs, in February 1933, with a level corresponding with 471·4 at the barrage site. The depth of water on this occasion may be assumed at 2·0 feet, giving a bed-level of 469·4. From a survey of the river made in the winter of 1936–37, the highest minimum bed-level downstream of the barrage site was found to be 469·0, which agreed fairly closely with the above and was accepted as the level of the subsoil water likely to occur under normal river conditions on the downstream side with the barrage closed.

In 1931 two irrigation engineers in the Indian service published a very valuable report [1] dealing with the effect of the construction of weirs and canal-withdrawals on river regime. An important point commented on by them is the retrogression of the river-bed which occurs downstream of a barrage in the first few years after its construction. This may be explained by the fact that the heading-up at the weir, and the resulting flatter slope in the river upstream, causes the water to drop some of its silt; downstream of the weir the relatively silt-free water erodes the river-bed and there is consequently a fall in specific levels. (A specific level is that corresponding with a given discharge.) The effect is purely temporary, since as soon as the pond above the work silts up and provides a normal river section, the normal silt charge, and consequently the normal slope and levels, are restored to the water crossing the weir. A period of 15 to 20 years is required for the cycle of changes to take place, and in the meantime the low water-levels may have serious effects, since the exit gradient of subsoil flow will be steepened and the cistern-depth reduced. It is necessary, therefore, to provide for the low levels which will occur during this period. From the gauge records at the various project canal-headworks, it has been determined that river retrogression of this description may effect low river-levels to the extent of 4 to 7 feet. With high discharges the effect is not felt so much, since in high rivers the water overflows its normal bed and occupies a section which is little affected by this process. Retrogression of 1·0 to 1·5 foot has, however, been observed in high specific gauges.

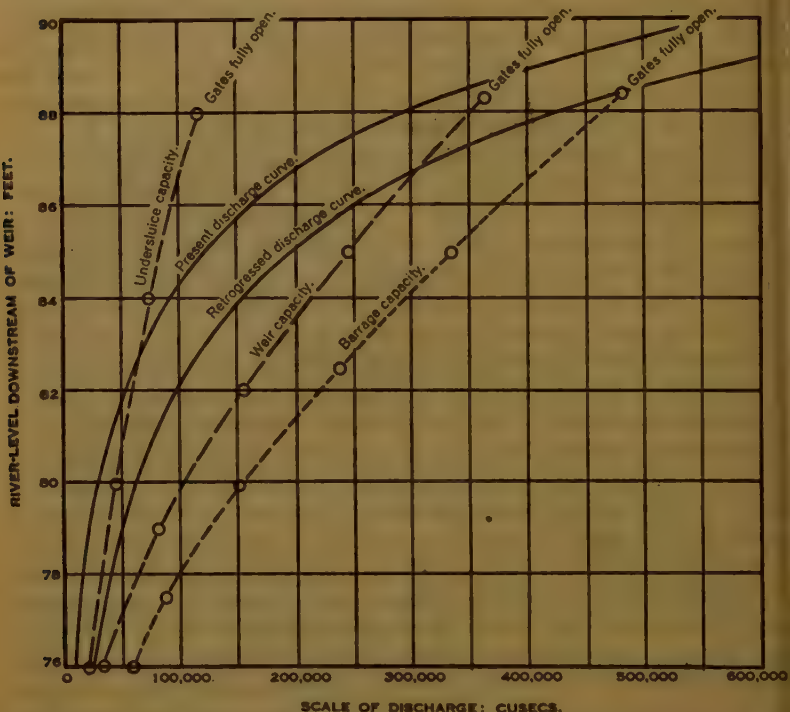
In designing the Emerson barrage, provision was made for a retrogression varying from 5 feet for no discharge to 1 foot for the maximum flood discharge namely, levels of 464·0 and 489·5 respectively. Discharge curves for the normal and retrogressed conditions are given in *Figs. 8*.

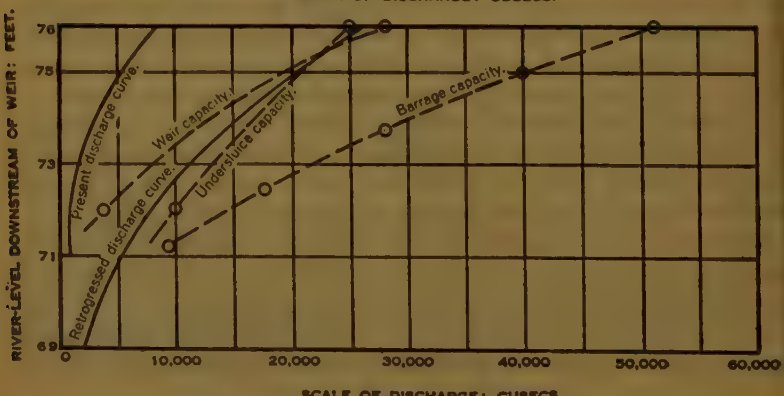
The length of the Emerson barrage was largely based upon experience at other headworks, using as a criterion Lacey's formula $P_w = 2{\cdot}67\sqrt{Q}$, in which, for a wide shallow stream, such as are all large rivers in flood, $P_w$ may be taken as the surface width. Table I gives the width between abutments, $B$, the maximum flood discharge, $P_w$, and the ratio of $B$ to $P_w$

[1] H. W. Nicholson and W. L. C. Trench, "Report of the Committee of Superintending Engineers on the probable effects of the Bhakia Dam Scheme on the inundation canals of the Indus." (1931.) The Civil and Military Gazette Press, Lahore.

for various Punjab and other Indian headworks, and also the figure[s]
adopted for Trimmu.

*Figs. 8.*




BARRAGE DISCHARGE CURVES.

The width was kept on the high side in view of the fact that the cos[t]
of the section increases rapidly with the discharge per foot run.

TABLE I.

| Headworks. | Discharge. | $P_w$. | $B$. | $B/P_w$. |
|---|---|---|---|---|
| Ferozepur | 450,000 | 1,780 | 1,956 | 1·1 |
| Suleimanki | 325,000 | 1,520 | 2,223 | 1·47 |
| Islam | 275,000 | 1,400 | 1,621 | 1·15 |
| Panjnad | 700,000 | 2,230 | 3,400 | 1·56 |
| Sarda (U.P.) | 600,000 | 2,070 | 1,964 | 0·95 |
| Sukkur (Sind) | 1,500,000 | 3,260 | 4,725 | 1·45 |
| Balloki | 113,000 | 930 | 1,547 | 1·67 |
| Trimmu | 645,000 | 2,140 | 3,026 | 1·42 |

For economy in the cost of gates and piers, it is essential that the barrage crest should be kept as high as possible. Flow on the gate section should be critical, and the depth of the crest below upstream total-energy level is then given by $q = 3{\cdot}1H^{1{\cdot}5}$ where $q$ denotes the discharge per foot run. Assuming one-tenth of the length between abutments to be occupied by piers, the average discharge per foot run of waterway was 238 cusecs, requiring a depth of 18·3 feet. With the moderately gentle expansion proposed in the downstream glacis, an afflux of 15 to 20 per cent. was considered necessary, for which 3·0 feet was adopted, giving an upstream average maximum flood-level of 493·5.

Command of the Haveli canal with a 1-foot head required a level upstream of the barrage of only 490·0, and the full supply level of the Rangpur canal was fixed at that which could be commanded by the same pond-level. The level of the top of the gates was, however, fixed at 493·0, thus rendering it possible, with a pond-level of 492·5, to store 15,000 cusec-days of additional supply. The capacity of the pond between levels of 490·0 and 486·0, the level necessary to command the maximum discharge of 2,750 cusecs permissible in the winter for the perennial areas, is 10,000 cusec-days. The total storage possible is thus 25,000 cusec-days which, after allowing for pond absorption-losses, will give a very useful addition to the limited winter supplies. This water can be stored at the end of the monsoon before the river falls to winter canal capacity, and also during any winter freshets which may occur.

The Emerson barrage is equipped with a set of low-level gates at each end. The purpose of these sluices is three-fold: (*a*) they render it possible to maintain, by scouring, a deep channel in the vicinity of the canal off-takes, in which low velocities of approach to the canals can be secured, thus facilitating silt-control; (*b*) they facilitate the diversion of the river over the completed barrage; (*c*) after completion, they facilitate the unwatering of the lower portions of the work for inspection and maintenance.

In the past Punjab engineers have been taunted with the fact that in practically all their headworks the width of the undersluices has been kept at 240 feet, irrespective of the size of the canal or river, and they have accordingly been accused of having no rational basis for the design

of these works. The accusation is largely true; even Khosla's workb which represents the latest ideas in the Punjab on the subject of wei design, makes no suggestion as to how the waterway or discharge should be determined. It would seem, however, that to attain the first object detailed above, the width of the sluices should be made proportional to and rather greater than, that of the offtaking canal, and that the leve and width should be suitable for the second purpose.

In the present instance the considerations which governed the undersluice length were as follows: the undersluice cistern-level was first fixed at downstream river bed-level under conditions of maximum retrogression, namely, 464·0, this being the maximum level which permitted the escape of low supplies without forcing the wave off the cistern floor.

With this floor-level, the maximum discharge with high flood-levels, still consistent with the depth on the downstream floor being sufficient to support the wave, was then calculated and the corresponding undersluice crest-level was determined. With this level (472·0) the total width of undersluices was made sufficient to take the possible discharge of the river (assumed as 10,000 cusecs) at the time of diversion with low afflux on the normal level corresponding with the discharge, that is, with a level of 476·0. This gave a discharge per foot of 24 cusecs and required a length of waterway of 417 feet, against which 420 feet was provided. Of this, the customary 240 feet was allocated to the left flank, leaving 180 feet for the right. These widths are suitable for the canal-bed widths of 175 and 144 feet respectively.

The maximum discharge which the downstream floor could take was determined by trial and error from the curves indicated in *Fig. 3*; but before this could be done it was necessary to calculate the total energy-level under flood conditions, since the recorded high flood-level is, of course, the actual water-surface. This was calculated by taking the average discharge per foot run, namely, $\frac{645,000}{3,000} = 215$ cusecs, and calculating the corresponding depths assuming the bed-silt to correspond with a value for Lacey's $f$ of 3·0, a figure purposely taken on the high side for safety. This gave a depth of 23 feet, a velocity of 9·4 feet per second, and a velocity-head of 1·38 foot. The high flood total energy-level was therefore taken as 495·0 upstream and 492·0 downstream. The corresponding discharge-intensity permissible on the undersluice floor was 341 cusecs, giving the crest-level mentioned above. It was also established that with a discharge per foot run 20 per cent. in excess of the normal, such as might result from the uneven distribution of discharge across the river, namely, 408 cusecs, with a total energy-level upstream of 498·0 and the same afflux, the wave would still be contained in the cistern. 30-foot spans were decided upon for the undersluices, as being suited to the great height of the gates, whilst past experience had shown 60-foot spans to be suitable for the probable height of the weir gates. After allowing for 7-foot piers between bays and 25 feet

n each side for the divide-wall and fish-ladder, thirty-seven 60-foot pans were found to be suitable for the weir. The high flood discharge per foot run of weir with the undersluices fully open works out at 226 cusecs, requiring a total energy-depth of 17·5 feet and, consequently a crest-level of 477·5. With a 20-per-cent. concentration of supply, that is, a discharge of 271 cusecs per foot, the maximum height of cistern-floor permissible was 470·0. There were, however, various reasons which made it desirable to keep the floor lower. For example, with the maximum head of 29 feet on the barrage and a water-level of 464·0 downstream of the undersluice pockets, spring level would certainly be lower than 470·0 downstream of the weir in the vicinity of the undersluices. The head on the work is, of course, the difference between the upstream level and spring level, and the exit gradient depends upon the depth of cover of the downstream pile-line. With a high-level floor the necessary low gradient can be secured by increasing the depth of the pile-line, but in this case the pressures under the floor are increased when the downstream water-surface is at floor-level. Again, a lower downstream floor increases the flexibility of control of the barrage when in operation, permitting a given supply to be passed through a fewer number of bays. From these considerations, and in view of the fact that, with the low levels necessary for the unwatering of the pockets during construction, little extra expense was involved in lowering the weir floor, the level of the latter was fixed at 468·0. A spring level of 466·0 was, however, allowed for in considering the exit gradient.

The flexibility of control of the barrage in operation with low supplies was studied by means of the curves of *Figs. 8*. In these Figures are plotted the normal river discharges; the river discharges under assumed conditions of maximum retrogression; and the capacity of (*a*) the undersluices, (*b*) the weir, and (*c*) the barrage; with an upstream level of 493·0, as limited by the condition that the downstream depth should be sufficient to support the wave.

The proportion of discharge to capacity at any level indicates the minimum proportionate length of the work required to be used.

*Design of Sections.*

In accordance with the modern theory of weir design, it is necessary to provide for the safety of the works under dynamic action and under the effects of subsoil flow.

*Dynamic Action.*—Under this heading it is necessary to consider the following points:

(*a*) The downstream floor must be sufficiently low to ensure that the standing wave will form on or upstream of it.

(*b*) The length of floor at this level must be sufficient to ensure that it will underlie the length of the wave, in order that the violent action thereof will be contained within an inerodible boundary.

(*c*) Upstream and downstream of the work the river-bed is liable to

scour to considerable depths below the level of the weir floor. In ordd to guard against such scour occurring close to the permanent work, and thus undermining it, a length of flexible protection is provided upstream and downstream, designed to resist erosion, but to settle when undermined and thus provide an inerodible covering from the edge of the weir to the bottom of any scour-hole which may occur. Usually this protection consists of stone, and the quantity used must be suited to the probable depth of scour below the floor level.

(*d*) In the case of the upper portion of this protection the velocity may be high enough to move stone by erosion. If this be so, the upper portion of the protection must be formed of blocks of sufficient size to resist the velocity, arranged so as to settle without dispersion.

(*e*) Under the trough of the wave the static pressure above the floor is reduced below that corresponding with the downstream water-level. The pressure under the work under these conditions of flow can be calculated, and the floor should be heavy enough to withstand the resulting difference in pressure.

*Effects of Subsoil Flow.*—To guard against the effects of subsoil flow:

(*a*) The gradient of subsoil flow at exit must be low enough to avoid any danger of "piping."

(*b*) The floor must be sufficiently heavy to resist the subsoil pressure beneath it.

(*c*) Since the safety of the work against "piping" depends upon the maintenance of the cover downstream of the downstream pile-line, it is necessary to provide for this by means of a permanent porous floor (blocks over an inverted filter), at this site.

Of these factors, the level of the downstream floor has already been considered.

In accordance with the theory enunciated above, the length of cistern floor was fixed at five times the height of the wave. The latter was obtained from the curves indicated in *Fig. 2*, using the figures of concentrated discharge per foot run of 271 and 408 for weir and undersluices respectively. The lengths, as determined by this means, were fixed at 66 and 80 feet.

For calculating the probable depth of scour upstream and downstream of the work, the silt was assumed to correspond with Lacey's $f$ equal to 1·0 a low figure for the site. The discharge per foot was calculated on the waterway between abutments, including piers, with the usual 20 per cent added for lack of uniformity. The probable depth of scour was obtained by increasing the normal depth upstream by 25 per cent. and downstream by 50 per cent. The resulting bed-levels were:—

| | | |
|---|---|---|
| Undersluices . . . . . . . | 442 upstream. | 427 downstream. |
| Weir . . . . . . . . . . | 453 „ | 440 „ |

The level to which the floor was taken down on the upstream side wa

letermined, from a consideration of the probable difficulty of unwatering, ıt 470 and 468 for weir and undersluices respectively.

The velocity which was considered unsafe for stone was taken as 10 feet per second, and the levels above which this velocity was likely to be exceeded were ascertained to be :—

| | | |
|---|---|---|
| Undersluices . . . . . . . | 466 upstream. | 462 downstream. |
| Weir . . . . . . . . . | 471 ,, | 468 ,, |

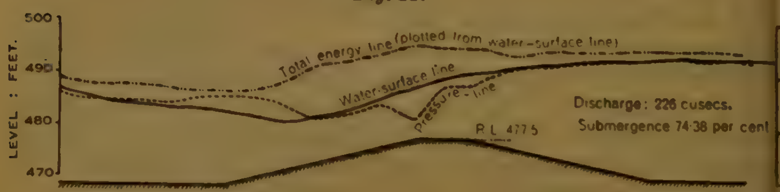
The block protection provided is thus ample to guard against scour in the vicinity of the floor. The stone provided was calculated as sufficient to cover a slope of 1 in 3 from floor-level to the level of deepest probable scour to a thickness of 3 feet.

Before discussing the design of the section against subsoil flow effects, t is considered advisable to explain how the length and surface profile were obtained. It was obvious, of course, that from the upstream side the profile should start from a level low enough to ensure a reasonable velocity on the upstream flexible protection, should then rise to the crest, which should be level for a sufficient length to accommodate the gates, and should then fall to the cistern floor-level. It was evident, also, that the rise should form a gentle contraction of the waterway and that the crest, should be relatively narrow if a high coefficient of discharge was to be secured. Similarly, on the downstream side a gentle expansion was desirable to reduce energy-losses, and hence afflux, under high flood conditions. Before deciding on the exact form of the profile, model-experiments were carried out in the Irrigation Research Institute on profiles embodying slopes on the upstream and downstream side varying from 1 in 5·7 to 1 in 3, and slopes connected to the level portion of the crest with circular curves. The Author had anticipated that the latter type of profile, affording a very gentle change of section of waterway in the vicinity of the control point, would give a relatively high coefficient and low expansion losses. This, however, was not found to be the case, there being no appreciable difference between the profile comprising simple slopes and those embodying curves. The various models were tested for coefficient of discharge, effect of submergence, and downstream erosion. As regards the former two points, no important difference was observed. It was established, however, that a coefficient of 3·2 could be expected on a section with end-contraction suppressed, which corresponded well with the 3·1 assumed in the design, and that the afflux assumed was ample. As regards the last point, it was found that a steeper downstream slope was associated with less erosion. It was considered, however, that a slope steeper than 1 in 4 would present constructional difficulties, and consequently this slope was decided upon both upstream and downstream. These slopes, associated with the length of cistern, and the levels of crest, and upstream and downstream floors, gave a total length of work of 140 feet in the case of the weir, a length which was also found to give a suitable

exit gradient. In the case of the undersluices, the length of the floor wa taken at 200 feet. Actually, as the pockets are paved for 240 feet upstrea of the gates, a greater length might have been assumed. As the pavim is only 1·5 foot thick, however, it was not considered advisable to conside the whole of it as forming part of the undersluice section. A pile-line wa introduced at a distance of 200 feet from the downstream end of the floo: the paving was thickened below this line, and this 200 feet was taken as th length of the floor when considering its strength against subsoil flow.

After the general profile of the section had been decided, further test were carried out in the laboratory to study the comparative merits of di ferent systems of dissipator blocks on the glacis and floor in reducing erosio downstream, and as a result the arrangement shown in Figs. 9 and 1( Plate 1, was adopted. The actual hydraulic pressures on the glacis an floor in the vicinity of the wave were also studied, and were found to diffe considerably from static. This investigation was undertaken at th Author's request in order to test his contention that, as the jet is deflecte

*Fig. 11.*




from inclined flow to horizontal at the foot of the glacis, a considerabl pressure must be exerted upon the floor, which is useful in counteractin the low static pressure in the trough of the wave. The existence of thi pressure was confirmed, as is evident from the diagram reproduced a *Fig. 11.*

Both weir and undersluices were provided with lines of steel sheet piling, at the upstream end, to guard against any danger of dynamic actio undermining the floor in the case of the flexible protection failing; at th downstream end, to provide a safe exit gradient; and at the upstream en of the cistern, to reduce the uplift pressure under the latter.

Moreover, in the case of the undersluices, a fourth line was provided a the toe of the upstream glacis to act as a reserve in case of failure of th comparatively light upstream floor.

The depth of the downstream line of piling was made sufficient t ensure an exit gradient safe against "piping", using the upper curve i *Fig. 5*, p. 117. For example, in the case of the weir, the depth of th piles below the bottom of the filter is 18 feet, from which the valu of $\frac{1}{\alpha}$ is 0·128 and the factor $\frac{1}{\pi\sqrt{\lambda}}$ is 0·153. The exit gradient fo

he maximum head of 27 feet, $G_E = \frac{H}{d} \frac{1}{\pi\sqrt{\lambda}}$, is thus 0·23 at this level.

There are no very clear principles governing the depth of the inter-nediate and upstream piles. The purpose of the former is to reduce the pressure under the cistern, and also to act as a second line of defence in the inlikely event of the downstream line failing. For the former purpose, a lepth equal to that of the downstream line is suitable. Additional depth n this position results in little reduction of the pressure, and this depth is sufficient to give a suitable exit gradient should the downstream line fail. In this event, the head over the weir would be strictly limited until repairs vere effected.

The depth of the upstream line should be based upon the probable lepth of scour. It seems unnecessary to take it down below the corre-sponding bed-level in order to provide cover to hold the bottom end of the pile against failure by spewing, as the loose protection upstream of the veir after settlement will always prevent the full depth of scour occurring n the immediate vicinity of the pile-line. It was therefore considered sufficient to take the piling down to the level of the deepest probable scour.

Minor departures in the completed work from the depth of piling heoretically required are accounted for by the fact that use had to be nade of a quantity of piling of fixed length available locally.

All the main lines of piling were continued for an adequate distance nto the flanks, and cross-lines were provided at the abutments, at the livide-walls, and on four intermediate sections.

In order to ensure adequate cover for the downstream lines of piles t all times, a special pressure-relief area was provided between this cut-ff and the downstream flexible protection, the width of which was mpirically made equal to the depth of the piling. In this area, security gainst dynamic action was attained by a surface layer of 5-foot by 3-foot locks, 4 feet deep, whilst the danger of "piping" was avoided by pro-iding beneath this an 18-inch layer of 2-inch stone ballast on a 6-inch ayer of coarse sand. The ballast merely acts as a French drain to permit he passage of the subsoil flow to the joints between the blocks, which were nade 3 inches wide, and filled with ¾-inch ballast. If this 2-inch ballast ere placed directly on the natural river-bed, it would sink into the bed or a few inches and its interstices would be filled with the bed sand. The ubsoil flow emerging from below the weir would find its path constricted y the presence of the ballast, and a high pressure-gradient would be estab-shed through the sand in the ballast interstices. "Piping" might then e caused, which would be progressive to failure. By introducing under he ballast a layer of coarse sand, with a transmission-constant about ten mes that of the bed sand, this danger is avoided, because, as the pressure-adient varies inversely with the transmission constant, though it will ill be locally intensified, it cannot attain a dangerous figure. The

pressure-relief area is separated from the downstream flexible protectio by a curtain wall 8 feet deep.

When the filter is first constructed, and so long as it remains unchoke the loss of head in the seepage-flow through it is negligible, and the ex gradient must for safety be calculated at its under side. The filter, how ever, may, and usually does, rapidly become choked by silt deposited fro the river flow above it. In this event, a steep gradient will be establishe in the filter itself. This does not matter from the point of view of th exit gradient, because although it may blow out the deposited silt, suc movement cannot be progressive beyond the bottom of the filter. Thi action has, however, a disadvantageous effect upon the pressures under neath the floor, since the pressure at the bottom of the filter may b appreciable and will, of course, increase the pressure at all points upstrea in the seepage-flow. In calculating floor-pressures, therefore, the down stream cover should be considered to extend at least to the top of the filte and it would be safer, and more logical, to assume an extra depth of cove equal to the depth of the filter to provide for the low transmission-consta of the latter when blocked. This was not done, however, in the design o this work.

As the Emerson barrage was the first important work to be designe in the light of the modern knowledge of seepage-flow, the safety factor t be allowed in the exit gradient required careful consideration. That final adopted was 5·0, calculated at floor-level, that is, assuming the filter t have the same resistance as the underlying bed. Calculated at bed-leve the corresponding factor was about 4·5.

Although the older Indian headworks were not designed on the moder theory, it is possible to calculate the exit gradients which are provided i existing works, and consequently their safety factor. The followin Table gives figures for a number of such cases :—

Table II.

| Work. | Maximum head: feet. | Safety factor. | |
|---|---|---|---|
| | | At floor-level. | Below filter. |
| *(a)* Works with downstream cut-off sheet-piles. | | | |
| Sukkur | 24 | 5·1 | 4·1 |
| Panjnad annexe | 23 | 5·4 | 4·5 |
| Khanki, Bay 8 | 23 | 5·1 | 4·4 |
| Marala | 14 | 4·4 | 4·2 |
| *(b)* Works with shallow curtain-walls only. | | | |
| Islam | 21 | 3·7 | — |
| Suleimanki | 20 | 3·3 | — |
| Ferozepur | 22 | 3·5 | — |

It is also useful to compare the safety factors commonly used for other materials. For steel the usual figure is 4·0 on ultimate strength, or 2·5 on elastic limit, whilst for concrete the safety factor is usually about 3·0. Both of these materials are, however, of a very homogeneous nature, and for a less reliable material, such as wood, a factor of 6·0 or 7·0 is commonly adopted.

At first sight it would seem that sand can only be classed as an unreliable material, but a little thought will show that this is not so. In the case of the materials mentioned above, the safety factor is intended principally to cover variations in the strength of the material in tension, compression, or shear, and is based upon the variations in strength of individual specimens, which are revealed by experiment to be considerable. In the case of the exit gradient, however, the criterion is not strength, but weight, and it is known that the variations in specific gravity are negligible. On the other hand, variations in gradient, caused by lack of uniformity of the materials of which the river-bed is composed, have to be allowed for. The high degree of agreement between pressures recorded in the field and those calculated by theory shows, however, that large variations are impossible from this cause, at all events in an alluvial river-bed known to consist of pure sand. On the whole, therefore, the safety factor provided seems ample.

The high heads to be provided against and the long low-level cisterns on the downstream side of the barrage, involved unusually high uplift pressure under the floor. Two alternate methods of design were possible: to provide sufficient thickness of floor to resist these pressures by weight directly, or to utilize the weight of the piers, and where this did not exist, to provide sufficient weight in groynes, and design the floor as a beam to transmit the pressure to the piers and groynes. For a gravity section, the floor masonry would have had to be as much as 12 feet thick in places, and the difficulty and cost of unwatering the foundations to the required level would have been very considerable. The extra thickness would also have made an appreciable difference in the total quantity of ballast required for the work, and in the time in which it could be quarried and brought to site. On the other hand, misgivings existed regarding the probable quality of the reinforced concrete likely to be produced in these very deep and wet foundations, with fears of premature rusting of the reinforcement owing to water gaining access to it through temperature- and setting-contraction cracks.

The section finally adopted was a compromise between the two methods of design. The thickness of the cistern floor was limited to 3·5 feet, which did not involve excessive unwatering but provided sufficient weight to resist dynamic action, and sufficient reinforcement was included to carry the excess pressure to the piers and groynes. In order to guard against unsound concrete, a 9-inch course of 1 : 3 : 6 concrete was laid under the whole of the reinforced floor, the weight of which was ignored in the cal-

culations. Sufficient reinforcement was provided to guard against anything other than hair cracks resulting from temperature- and setting-stresses. Whilst it was not assumed that water would not reach the reinforcement through the hair cracks, in view of the fact that the floor will almost invariably be submerged, the danger of rusting was regarded as being very remote. In order to reduce the bending moments, and consequently the reinforcement, in the floor in the 60-foot spans, central groynes were introduced. The height of these groynes was limited to crest-level, namely, 9·5 feet, in order that they might offer no obstruction to the flow over the weir, and their width was restricted to that of the piers, as in the case of the groynes in continuation of the latter. To obtain the requisite weight, the pier groynes had to be made rather heavier than the intermediate groynes, and the loading of a half-span is thus asymmetrical. The floor is designed as a continuous beam throughout the length of the weir, no provision being made for expansion joints. Owing to the great thickness of the floor, considered as a beam, the compressive stresses are unusually low, the maximum being 500 lb. per square inch. Advantage was taken of this fact to economize in the cost of the concrete by using a 1:2½:5 mix. Experiment showed that the concrete made from the unusually heavy Sikhanwala aggregate had a specific gravity ranging from 2·6 to 2·7, and hence, in comparison with the figure of 2·4 assumed in the design, the floor may be said to have a margin of weight of about 10 per cent. in excess of the uplift pressure.

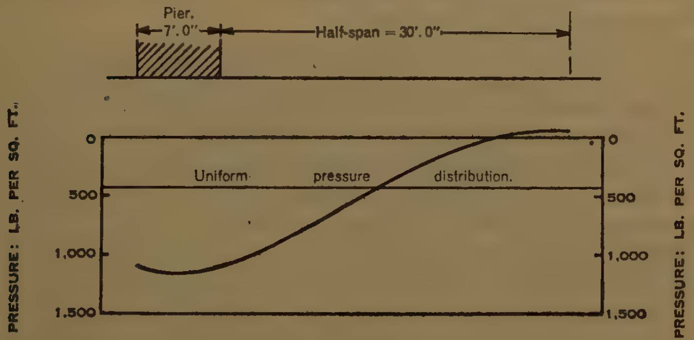
Some doubt was expressed as to whether the intermediate groynes would increase the turbulence downstream of the work, it being feared that large eddies might be set up by them. The matter was investigated on a model at the Malikpur field station of the Research Institute, and it was demonstrated that downstream scour was distinctly less with the groynes than without them.

The piers of the weir and undersluices carry a 20-foot road bridge and the gates, with their counterbalances, gearing, and superstructure. Consequently their weight is considerably in excess of the uplift pressure under the section on which they stand. To support these piers, there was the alternative of providing independent foundations or of carrying their weight on the floor. For independent foundations, open construction was out of the question, owing to the difficulty of unwatering, and the cost of wells made that method uneconomical. Independent foundations were also difficult to combine with the inverted-beam design of floor. The floor would have had to be constructed after the piers were complete and loaded, or excessive load would have been transmitted to the floor.

The possibility of constructing the floor first, leaving an open joint to be grouted after the piers were complete, was realized, but this would have led to delays in construction, necessitating the return to low-level work at a late stage. A gravity-section floor in the upper part of the work was also considered, but was rejected on account of the difficulty of obtaining a

satisfactory joint between it and the beam section of the cistern. It was decided, therefore, to design the floor as a raft-foundation for the piers. As an additional precaution, the piers were carried down underneath the floor to a line joining the bottoms of the upstream and downstream curtain walls, but no benefit was assumed from this in calculating the floor strength. The pressure-distribution at the base of the piers was, of course, calculated for the alternatives of open gates and wind on the downstream side, and for full wind and water pressure on the upstream side. The calculation of longitudinal distribution of pressure under the floor was based upon the analysis given in an article on shallow foundations by W. S. Gray[1], the basic assumption of which is that the soil reaction is that of an elastic material. A typical pressure-distribution curve resulting from this method is given in *Fig. 12.* At the end of the work, the abutments and

*Fig. 12.*




retaining walls were designed as cantilevers anchored in the floor, which was thickened locally to provide the necessary strength.

In the section which supports the piers, the floor thickness is that of an economical reinforced-concrete beam continuous throughout, the maximum compressive working stress being 630 lb. per square inch, and the mix being 1 : 2 : 4.

An interesting point in connexion with the design of the barrage sections was the investigation of the strength to resist bending required in the pile-lines, a knowledge of which was required in order to enable the section modulus of the piling used in various localities to be specified. This was determined by considering the bending moment to which the pile is subjected when on the point of failure by spewing, the cover being assumed to have been reduced by scour to the level which would permit this to occur. In this case, the pile is in equilibrium under the following forces :—

(*a*) the reaction of the floor at the upper end ;

[1] W. S. Gray, "Shallow Foundations." *Concrete and Constructional Engineering,* vol. xxxi (1936), p. 280, May 1936.

(*b*) the bending moment due to the upper end being fixed ;

(*c*) the earth-pressure at the back of the pile, allowing for (1) a surcharge sufficient to balance the difference in subsoil water-pressure at the top and bottom of the pile ; (2) the buoyancy of the soil ;

(*d*) the water-pressure at the back of the pile, allowing for the pressure of seepage-flow ;

(*e*) the earth-pressure at the front of the pile, modified as in (*c*) ;

(*f*) the water-pressure at the front of the pile.

The effect of seepage-flow upon the earth-pressure is equivalent to increasing or decreasing the specific gravity of the soil by a figure equal to the pressure-gradient. For example, if the exit gradient is that of failure the soil can exert no pressure.

The condition of failure by spewing is found by trial and error for different depths of cover, and when this has been determined the maximum bending moment can be calculated.

The road-bridge provides a 20-foot roadway with 2·5-foot footpaths on each side and is designed for the heavy loading of the Indian Roads Association, namely, a distributed live load of 0·58 ton per linear foot, and a moving knife-edge load of 7 tons per traffic-line. This is equivalent to about 13 B.S. units of loading. As will be seen from the sections in Figs. 9 and 10, Plate 1, it is a reinforced-concrete T-beam structure stiffened by cross-members at the bearings and quarter-points. In the case of the weir, it is designed in sections continuous over one span and with a quarter-span overhanging on each side, two sections carrying a suspended 30-foot span. The suspended-span bearings are inclined and situated at mid-depth of the section, the load being transmitted through the end cross-members to the main T-beams. Excessive shear stress in the vicinity of the bearing is carried by extra steel. Provision is made for expansion at the bearings of the suspended span, and at alternate piers in the under-sluice where the spans are simple beams with ends free. The Haveli project will receive a credit from the Central Road Fund, representing roughly half the cost of the road-bridge, which will, of course, be open to the public.

The gates and gearing of the barrage and regulators were designed and constructed in the Irrigation Central Workshops at Amritsar at a cost of 1,800,000 *rupees*. The gates consist of $\frac{3}{8}$-inch skin-plates carried by horizontal bow-string girders to end frames which carry roller-paths pivoted to take up the gate deflexion under load. Fixed roller-paths are also provided in cast-iron grooves grouted into the piers and carried by machined sill girders which are grouted into the barrage crest. Live Stoney rollers operate between the roller-paths. The gates are suspended by steel wire cables running over spirally-grooved drums to steel counterbalance-boxes loaded with stone. The drums are carried on roller-bearings and are located in the framework of the superstructure below floor-level. They

are operated by spur-wheel gearing from centrally-located winches designed to be used by two men. The 60-foot gates are 15·5 feet high and weigh 24½ tons each. The 30-foot gates are in two sections, 9 feet and 12 feet high, weighing 11½ and 22½ tons respectively. The 12-foot section is mounted directly over the 9-foot section and operates in the same grooves, but on different roller-paths. Each section has its own gearing and counter-balance. The winches have 80-to-1 reduction gears, and the operating times for the weir and undersluice-gates are approximately 14 and 18 minutes respectively. All gates have suitable staunching devices, and the grooves have flanges protecting the rollers from water-action. The super-structure is of uniform height throughout, the barrage being 27 feet above the level of the road-bridge. The location of the drums below the floor leaves the operating platform almost entirely free of obstruction, and the design is of particularly pleasing appearance.

*The Pockets.*

The layout of the pockets is illustrated in Fig. 13, Plate 2. For scouring purposes, the undersluices may frequently be operated with high discharges while the weir remains closed. When this is done with moderate supplies in the river, deep channels will form from the main river-channel to the pockets on each side, running more or less parallel to the face of the work. One object of the divide-walls separating the weir from the pockets is to keep such channels away from the upstream face of the work and thus avoid damage to the upstream flexible protection. The nose of a divide-wall is usually the site of heavy action as the stream turns round it, and the length of the wall must be such that this action will not en-danger the upstream protection of the sluices, and that normal flow and silt-distribution may be restored before the water reaches the canal regulators. For this purpose, a length of 360 feet was considered sufficient for the main upstream divide-walls, whilst the central divide-wall in the left pocket was made 560 feet in length, that is 304 feet from the silt-excluder slab, in view of the importance of securing quiet flow upstream of the silt-excluder. On the downstream side the divide-walls were given a length of 405 feet, that is, about 275 feet from the downstream end of the weir pressure-relief area, in order to guard against damage to the weir flexible protection owing to action set up by the undersluice dis-charge. The difference in level between the weir and undersluice floors is adjusted by sloping the block and stone apron from the floor-level of the weir to that of the undersluices on the weir side of the divide-walls. The noses of the upstream weir divide-walls are sloped, since this form spreads the turbulence and reduces the scouring action on the bed. In the case of the central divide-wall, the nose is vertical, in order that the turbulence may be allowed as long as possible to subside before reaching the regulator, whilst on the downstream side the noses are kept vertical, to locate the action as far downstream as possible. The noses of the divide-walls

are founded on wells sunk to a maximum depth of 40 feet. The shanks have open foundations ending on the river side in a shallow curtain wall on a 10-foot line of piles. Erosion is guarded against by an apron of blocks, 35 feet wide, and by stone 70 feet by 5 feet at the nose and 50 feet by 4 feet on the flanks. The paving of the pockets was decided upon, partly because it was cheaper than a double line of flexible protection, and partly, more particularly in the case of the excluder section, because the smooth bed would prevent the lodgment of any obstruction likely to interfere with silt-distribution. The divide-walls are kept 7 feet thick with a plinth formed by a 12-to-1 batter (Fig. 14, Plate 2). They are reinforced vertically, and are designed to withstand a difference in level of 5 feet between the two sides. They rise to a level of 495·0 on the upstream and 492·0 on the downstream sides, that is, 1·5 foot above high flood-level.

The pocket paving is 2·5 feet thick in the portion which is considered to form part of the undersluice floor and 1·5 foot thick above this, except within 30 feet of the boundaries, where it is 2 feet thick. It terminates in a curtain wall and shallow pile-line. The normal protection of blocks and stone is provided upstream of this line (Fig. 10, Plate 1).

*The Regulators.*

Provision is made in both canal regulators for additional water for use in silt-extraction, the extra discharge being 2,200 cusecs and 1,000 cusecs for the left and right canals respectively. The regulator capacities thus become 7,300 and 3,700 cusecs. In the case of the Haveli canal the sill-level was fixed at a level low enough to permit low winter supplies to be taken without serious loss of head, thus allowing the pond to be drained to the lowest possible level, and maintained at that level, minimizing pond-absorption-losses. With a discharge of 1,000 cusecs the canal-level will be 482·7. The crest-level adopted is 481·0, and the five 24-foot spans will discharge 1,000 cusecs with a pond-level of 483·0. This waterway is liberal for the full supply discharge giving, with a pond-level of 490, a coefficient of 2·24 in the formula $q = CH^{3/2}$.

In the case of the Rangpur canal, winter conditions have not to be allowed for. Three spans of 24 feet are provided, with a crest level of 483·5. This will just take the future canal supply of 3,700 cusecs with a 490·0 pond-level. The orifice soffits are at a level of 489·0 for both canals, assuring unsubmerged flow with the normal pond-level of 490, and a reinforced-concrete breast wall rises to a level of 499·25, giving a freeboard of 5·75 feet above river flood-level. The piers, which are 5 feet thick, are extended upstream as streamlined vanes, the object of which is to divert the flow into the regulators with minimum loss of head.

The cisterns of both regulators are placed at river-level 477, the bed-level of the Haveli canal, which is low enough to support the wave under all practical conditions. The transition from the crest to the cistern has a

parabolic profile, which gives a high discharge-coefficient for highly-submerged conditions, and a jet sharply inclined at entry into the cistern for high-head operation. The latter condition leads to rapid dissipation of energy in the cistern, with little turbulence below it. Dissipator-blocks are also provided.

The safety of the regulators against subsoil-flow effects is provided for in the same way as in the case of the weir. In calculating the exit gradient and subsoil pressure, the pocket paving is assumed to have failed. A pile-line is provided upstream and a curtain wall 10 feet deep downstream. The object of the former is partly to guard the regulator foundations against undermining in the case of failure of the pocket floor, and partly to reduce pressure under the regulator floor: the curtain-wall is of gravity section reinforced only against temperature stresses. The downstream cut-off is designed to give a safe exit gradient of 0·25.

Both regulators carry public road-bridges of the same width and strength as the barrage. A section through the Haveli canal regulator is shown in Fig. 14, Plate 2.

*Silt Exclusion.*

During recent years increasing use has been made in the Punjab of silt-excluders and extractors based on the design published by Mr. F. V. Elsden [1] in 1922. These devices depend for their efficiency upon the fact that, in a flowing stream, silt tends to fall by gravity to the bottom, and consequently the quantity of silt in unit volume of water, or silt-intensity, increases with depth below the surface. This fact has long been known, and in old canal-regulators, attempts were made to utilize it by designing them to take top-water as far as possible, the sill being placed at a high level, and rising sills being provided to ensure this condition of entry at different river-stages. The raised sill is, however, of very little use in a large work, as the bottom water of the portion of the river next to the bank must enter the canal unless other means of disposing of it can be applied. Elsden's design effects this by providing tunnels under the regulator sill through which the bottom water can escape. Following this principle eight large silt-excluders have been constructed recently on Punjab canals, which are described in a Paper [2] by the Author. The behaviour of these works has been carefully watched by local officers and by the Research Institute, daily routine observations of silt-intensities above and below the works being made. From these observations it has been established that the excluders keep out from 30 to 70 per cent. of the silt approaching them. It has also been found that the smaller the proportion of the escapage to the canal supply the greater the efficiency of unit

[1] F. V. Elsden, "Irrigation Canal Headworks." Punjab Irrigation Branch Papers, No. 25. (1922.)

[2] F. F. Haigh, "Silt Excluders." Minutes of the Proceedings of the Punjab Engineering Congress, paper No. 211, vol. xxvi (1938), p. 53.

escapage, that is, the silt-intensity increases very rapidly in the vicinity of the bed. From this it follows that if the supply available for escapage is limited, it is better to divide it between two or more excluders than to use it all in one.

It is also established, from a comparison of existing works, that, as might be expected, the efficiency is increased if the supply approaches the excluders through a straight channel with bed and banks free from obstructions. A channel of this description of sufficient length will, of course, enable the optimum condition of silt-distribution to be established. Such an approach channel can easily be arranged by locating the excluder in the canal, although in this case some extra expense is involved in providing additional capacity in the canal and its head regulators, for the escapage-supply. A canal excluder (better termed an ejector) is, however, a comparatively light work. The alternative is to place the excluder upstream of the head regulator in the undersluice pocket. This can be done at low cost by using the " Khanki " type (so called because it was first used at the Khanki headworks), which has been adopted for the Rangpur canal. This consists of a triangular slab at regulator sill-level, in the space between the regulator face and one or two bays of the undersluices. This covers tunnels founded on the pocket floor, which run parallel to the regulator-face and discharge through the undersluice gates, which are used to regulate their supply. The width of the slab is made sufficient to accommodate a discharge of about half the canal capacity. An excluder of this type, however, is not very efficient, since the concentration of the silt in the lower layers of the stream is bound to be seriously disturbed when the latter turns into the regulator. Moreover the river channel in and upstream of the pocket is rarely straight and free from obstructions. At high river stages, too, the pocket flow is usually turbulent, and this is the time when silt-exclusion is most required. Such an excluder will, however, prevent the entry of coarse rolling silt and of some of the suspended silt. It also keeps out of the canal submerged debris, brushwood, etc., which its large tunnels can accommodate without difficulty, whilst a canal ejector, with its smaller orifices, could not deal with such debris without a trash-rack.

An excluder of the Khanki type was considered sufficient for the right pocket, as the Rangpur canal flows through country with a good slope and is amply provided with falls. Consequently there is no reason to anticipate difficulty in securing a slope suited to the grade of silt likely to be taken into the canal. Should any such difficulty be experienced, an ejector can easily be constructed in the canal head-reach, provision for the escapage discharge having been made in the head regulator capacity.

The case of the Haveli canal is, however, different. Here the levels are such that it has been impossible to secure a slope flatter than 0·095 in 1,000 between Trimmu and Sidhnai, and there are no falls. As the result of the lining, the velocities are comparatively high, and the values of Lacey's $f$ range from 0·9 for 200 cusecs to 1·3 for full supply. Supply

vill, however, have to be headed up occasionally at the tail and inter-nediate sites to command the lower canals and distributaries offtaking on the way, giving conditions under which very low values of $f$ will obtain. It is necessary, therefore, to ensure that only the finest grade of silt will enter the canal, in order to minimize the silt-deposit resulting from these conditions, and so that when such deposit occurs, little difficulty will be experienced in subsequently clearing the channel with increased supply.

To effect this, an improved type of excluder has been provided in the right pocket, and two ejectors have been constructed in the canal, at 1,200 and 2,000 feet respectively from the head.

The pocket excluder is designed to eliminate the disadvantages of the Khanki type so far as possible. The slab covers the first four bays of the undersluices and its upstream edge is arranged normal to the river axis, thus ensuring the separation of the escapage before the stream turns into the regulator. The central divide-wall also provides a short length of straight approach-channel. This design, however, has the disadvantage that distribution of canal discharge over the slab cannot be controlled. It is obvious also that approach conditions will still not be so good as in the canal, and that the efficiency will be low when the undersluices on the left side are open. Notwithstanding this, the design should be an improvement on the Khanki type. The tunnels are 34 feet wide by 11·67 feet high (Fig. 14, Plate 2), and should have very little chance of ever being blocked by drift-wood. The design of the slab for this large span is interesting. It is not proposed to permit the condition of a high pond-level over the slab and the tunnels fully open to occur. Excluding this, the worst condition for which the slab is designed is that of the gates fully open with a high pond-level upstream, and a low river. The depth of water on top of the slab is that of critical flow at the downstream end, and will increase upstream at the slope necessary to maintain the discharge. Under the slab, the pressure will be atmospheric at the downstream end and will be increased upstream by the friction losses in the tunnel, which are calculated from Manning's formula.

The condition of flow with the tunnels closed and the upper gates open has also to be provided for. In this case, the resultant load is negative, requiring the slab to be tied down.

The slab is designed as a continuous beam in which a $1 : 1\frac{1}{2} : 3$ mix permits the use of the high compressive working stress of 1,100 lb. per square inch in the concrete, with the result that the slab thickness is reduced to the minimum, being 16 inches, with local thickening to 21 inches at the ends.

The canal extractors, being situated at 1,200 and 2,000 feet respectively from the head, have very good approach conditions, the canal sections being designed to give values of $f$ of 0·7 and 0·6 upstream of the first and second extractor respectively. The slabs are placed only 2 feet above bed-levels, in order to avoid any loss of head in the canal flow. They are

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triangular in plan and each covers four tunnels, 4 feet high, the width being 6 feet for the first and 5 feet for the second extractor. Each tunne serves a quarter of the canal waterway, and water enters them throug orifices in the upstream face wall. The extractors are designed to wor with a minimum head of 5 feet, permitting the escape of 1,200 and 1,00 cusecs respectively on this head. When they are fully open, the discharg is controlled by weirs situated downstream of the tunnels which will giv these discharges with a total-energy level 5 feet below canal full-suppl level. The discharge is divided equally between the tunnels, and conse quently the velocity is fixed at 12·5 feet per second. For this velocit the tunnel losses can be calculated, and the upstream orifices are pr portioned to give the designed supply under the balance of the head. Gate are provided at the downstream end of the tunnels for closing off an regulating the escapage. The extractors discharge through short tail-race into the river downstream of the barrage, and consequently have to b designed to work with low outfall-levels. The cisterns are low enough t retain the wave under all conditions likely to occur in practice. Th resulting level of the cistern floor is about 12 feet below spring-level, an the conditions of seepage flow are consequently rather severe since, wit the extractors closed and low levels downstream, flow will converge fro three sides into the bed of the tailrace. To meet these conditions, a lin of sheet-piling is provided on the three sides and under the wings, and system of pressure-relief consisting of an inverted filter discharging throug pipes and silt-traps is provided under the floor. Assuming that sprin level upstream of the pile-line will be 12 feet above the cistern floor, th exit gradient under the filter will be 0·25. The silt traps are intended t prevent choking of the filter by reverse flow on the rare occasions when th downstream water-level is above spring level.

*Training Works.*

The guide-banks of the Emerson barrage are of a modified Bell typ extending upstream to a distance of 3,480 feet measured along the rive axis. In plan the banks give a bellmouth approach to the barrage (Fig. 1 Plate 2). Starting from the gate-line, the left shank runs parallel to th river-axis for 1,500 feet, and then curves outward for 15 degrees 4299·3-foot radius curve and terminates in 90 degrees of 1,000-fo radius curve; the right shank runs parallel to the river-axis for 1,00 feet, then curves outwards for 26 degrees of 4299·3-foot radius curv and terminates in 90 degrees of 1,000-foot radius curve. At least tha was the original design; but subsequently small modifications in th total length of the barrage introduced a slight splay in the straig portions of the shanks. The distance between the centres of curvatu of the heads is 5,850 feet and that between the noses 6,830 feet.

There has been considerable controversy in the past as to the relati merits of bottle-neck, parallel, and expanding guide-banks. Sir Robe

ales[1], M. Inst. C.E., has recently stated the case for the first two types before 'he Institution. The decision to adopt the expanding type at the Emerson arrage followed from Sutlej Valley experience, where the use of the other ypes was not attended with success. At Suleimanki, where the bottleneck ype was used, it was found impossible to maintain the head of the right ank under strong river attack at reasonable expense. With this type, nd also with the parallel type as at Islam, it has been found that when a uide-bank head comes into action and the river is thrown across towards he other flank, a semi-permanent deposit is built up along the guide-bank hank which masks the approach to the canal, producing conditions of isturbed flow at the regulator. The use of the diverging type was trongly recommended by the committee appointed to inquire into the auses of the disaster to the Islam weir in 1930. Consequently it was dopted at Panjnad, where it has so far been successful.

Theoretically, the diverging type has two very great advantages in omparison with the other types; the heads, being farther apart, cover a arger width of the river-bed and are much less likely to be brought into ction by the lateral movement of the flow-channel, whilst, owing to their etirement from the barrage-flanks, they protect a much longer length of narginal bund. In view of these considerations, diverging guide-banks vere decided upon at Trimmu, and their exact alignment was fixed after series of experiments had been made on a model of the river at the Ialikpur field station of the Research Institute, where guide-banks of arious lengths and angles of splay were tried.

The guide-banks have a freeboard of 6·5 feet, and side slopes of 1 in 2 nd 1 in 3 on the face and rear respectively, the top width being 30 feet a the case of the shanks and 60 feet for the heads. The face is armoured vith pitching 2 feet thick, and stone aprons are provided, in which the uantity of stone is generally in accordance with the principles laid down n page 115, *ante*. On the shanks, however, the apron stone was limited o the quantity existing in the same position at Panjnad, which had roved to be sufficient. In the case of the heads of the right banks uptream and downstream, additional stone was provided, as the Malikpur xperiments had shown that these parts of the work were likely to be eavily attacked.

The left marginal bund (Fig. 15, Plate 2) is aligned parallel to the arrage until past the range of influence of the guide-bank head. It then ollows an economical alignment to high land on the river bank. At the arrage end it is kept 1,000 feet from the barrage line, in order to protect ne station area against waterlogging threatened from the high watervels which will prevail in the pond. On the right bank, the marginal

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[1] "The Principles of River-Training for Railway Bridges, and their Application to e Case of the Hardinge Bridge over the Lower Ganges at Sara." Journal Inst. C.E., l. 10 (1938–39), p. 136. (December 1938.)

bund is kept as close to the canal as possible, leaving only sufficient spa for the public road, and land for borrow-pits for the maintenance of th banks. It is aligned parallel to the barrage for a distance of 2 miles, which it crosses the old river-bed. Possibly in the future a spur may l required on this flank to protect the marginal bund in the reach which outside the range of influence of the guide-banks, but the construction this will be postponed until its necessity is demonstrated.

The marginal bunds have a freeboard of 6·5 feet over high flood-lev in the vicinity of the barrage, diminishing to 5·0 feet in the remoter reache The top width is 20 feet, with a 1-in-2 slope on the outer face. On th inner face, the slope is 1 in 2 down to flood-level, and 1 in 3 below i Where the bund crosses the old river channels, it is strengthened on th outer side to give a hydraulic gradient of 1 in 7 for the worst condition The bunds are protected against wave-action between the levels of 49 and 495 by spawl surfacing. Brushwood protection is also provided fro level 490 to the toe of the bank.

On the downstream side the canal-banks and station-area ring bun have a 3-foot freeboard over flood-level.

## Construction.

During the summer of 1937, work was mostly of a preliminary natur The site was 8 miles from the nearest railway-station at Muddoki, and broad-gauge line connecting this with the headworks was planned an constructed. Land was acquired for the station area and weir area, an the former was provided with a ring bund rendering it immune from sul mergence by the river floods of that season. The station-area layout wa planned in detail, comprising the railway-station with its various siding the power-house, workshops, locomotive-shed, godowns, and residenti and office accommodation. Construction of the workshops and powe house was commenced with bricks purchased locally.

A new quarry was developed at Sikhanwala. This consisted of a hi of hard laminar sandstone about 1½ mile long by ½ mile wide, rising to height of about 300 feet from the flat plain. The stone had a specif gravity ranging between 2·7 and 2·9, and was very suitable for concre aggregate and pitching stone, but unsuitable for building stone.

Sikhanwala railway-station lay opposite the hill at a distance of 1 mil The quarry layout was planned to give an output of two trains (about 40,0 cubic feet) of stone or ballast per day, which involved working the great part of the face of the hill. Broad-gauge lines were laid along both sid of the hill, with sidings to the various quarry-faces, duplicate weig bridges were installed, and sidings for marshalling the trains were co structed between the quarry and the station. A workshop, elabora watering arrangements, and quarters for the staff were also include The quarry output throughout construction was well up to expectatio

bout 12,000,000 cubic feet of material being dispatched in the 14 months rom January 1938.

*Machinery.*—A large quantity of reconditioned second-hand machinery vas available at very low cost.

The power-house was equipped with :—

Three 140-kilowatt diesel-electric sets generating at 3,300 volts.

Two 125-kilowatt diesel-electric sets generating at 3,300 volts.

These generators were second-hand and unreliable, and were supplemented by a number of portable steam and oil engines with a total output approximately equal to that of the power-station.

Railway rolling stock consisted of :—

Seven locomotives.

Three hundred and seventy 22-ton trucks suitable for open line working.

One hundred trucks suitable for internal traffic only.

Other plant comprised :—

Twenty-seven large centrifugal pumps, 8-inch to 14-inch.

Seventy-three small centrifugal pumps.

Four pile-drivers with No. 7 Mackiernan Terry hammers.

Twelve 1-cubic-yard concrete mixers.

Seven ½- and ¼-cubic yard concrete mixers.

Four air-compressors.

Four 2-foot gauge locomotives.

Five hundred 2-foot gauge tip-wagons.

Four dragline excavators of ½-cubic-yard bucket capacity.

Two 10-ton cranes.

*Excavation.*—Excavation was started in the weir area in September 937, and was carried out almost entirely by manual labour. Donkeys vere used to carry away the spoil, which was used as far as possible in orming a heavy ring bund required to protect the work from inundation uring the ensuing flood season, and was also trammed out for use on the uide-banks, marginal bunds, and canal banks where this could be done conomically. The dragline excavators were used only for clearing and eepening drains. Work was also pushed forward on the right guide-bank, hich it was intended to complete quickly so that it might serve to protect he river side of the ring bund from attack during the coming flood season. or the same reason an armoured spur was constructed on the river side f the ring bund. The total quantity of earthwork to be moved in the eir-area was 60,000,000 cubic feet, and this work was largely completed y May 1938.

*Pumping.*—A large creek of the river passed through the right side of he weir-area, and as soon as the river fell, after the 1937 flood season, ufficiently to render the operation possible, this was closed on the ring-und alignment; two large pumps operated by portable steam engines ere installed in the pond thus formed and pumping commenced. When

the excavation reached spring-level at a level of about 480·0, a central drai was constructed throughout the length of the work leading into this ponc and branch drains were provided to serve the whole area. Meanwhi sumps were being sunk over the whole area for the installation of addition pumping plant. The general pumping plan comprised three large sump 16 feet by 18 feet, one being situated downstream of each pocket and th third at the centre of the weir on the downstream side. These were supple mented by a number of 10-foot by 8-foot sumps spread over the work fc local control. The sumps were constructed of brickwork on reinforcec concrete curbs, and were sunk as wells, their bottoms being plugged wit concrete. The level of the bottom of the curtain-wall on the downstrean side of the pockets was 459·0, and to enable the water to be reduced to thi level the downstream-pocket sumps had to be sunk to a level of 451·( Each large sump carried three large pumps, whilst the small sumps usuall carried one or two of smaller size. The sumps were kept clear of silt b continuous dredging while the pumping was in progress. The pumps dis charged through a line of pipes, or locally-made sheet-iron troughin carried on scaffolding, on to the natural surface at a level of about 487·( and then along open channels into the above-mentioned creek downstrean of the ring bund. Arrangements were made for double-stage pumpin during the 1938 flood season, when it was anticipated that the outfa would not command the river-levels, but such pumping was never appliec A light bund on the ground between the main river and the creek whic joined the river about 8 miles downstream prevented access of river wate to the latter, the levels in which remained low.

In previous Punjab headworks construction, the unwatering of the wei area had been carried out in sections, and it is believed that this was th first occasion on which the whole area was unwatered simultaneously The pumped discharge was therefore large, ranging, with work in fu swing, from 40 to 50 cusecs and attaining a maximum of 65 cusecs durin the flood season.

*Sheet Piling.*—As the subsoil at the site is pure sand, the piling wor was straightforward and calls for little comment. The piling used was three kinds. Some Ransome piling, which was available within th Department, was utilized, whilst about 1,000 tons of Universal piling wa obtained second-hand from Sukkur; but most consisted of Larssen pilin imported specially for the work. Special piles for joining the various type junction-piles, and correction-piles were all manufactured locally. Th light Larssen piles were usually driven two at a time. In cases where was necessary to connect up two approaching pile-lines, this was done b using a pair of half-piles connected together by bolts working in slotte holes after bringing the two faces parallel by correction-piles.

The pile-drivers, four of which were used, were standard British Ste Piling Company equipment, and the hammers were of the Mackierna Terry No. 7 type. The shallow piling under the divide-walls was large

driven by a No. 3 hammer operated by a dragline excavator without guides.

The piling amounted to 250,000 square feet. Pile-driving was started in January and was substantially completed by May 1938.

*Concreting.*—The twelve large concrete-mixers were distributed, four on the upstream side of the weir, four on the downstream side, and two in each pocket. They remained in position throughout the work, and were brought into use as required. All were situated at the edge of the foundation pit, the mixer being at the low level and the storage bin on the natural surface, the side of the pit being revetted with sandbags to support it. The bin, which was 200 feet long, had a floor of dry bricks and a system of open drains for dealing with the water used in washing the ballast. A temporary godown for the storage of cement was located at the end of each bin. A broad-gauge siding served each of the bins and ballast and cement were unloaded direct into them. All ballast was washed by means of a hosed water-supply before use. Sand was obtained from the weir excavation, selected grades being carried and stacked in the vicinity of the mixers. A short length of tramway was installed in each bin, and the materials required for each batch were assembled and measured in tip-wagons, from which the mixers were charged. They discharged direct into tram-trucks, and the concrete was thus conveyed to the site. It was not, however, tipped straight into the work, but on to a sheet-iron floor nearby, whence it was loaded into mortar pans and placed in the work by hand. This method was found to give a much more uniform concrete than direct tipping.

The small portable mixers were used both to supplement the supply from the big machines, and to place concrete in positions not readily accessible, or where only small quantities were required. That for the road-bridge was placed partly from the small mixers working on the bridge itself, and partly from below. In the latter case, the concrete was at first lifted by hand, along inclined gangways connecting the weir floor with the over-bridge level. Later, however, the draglines were adopted for lifting the concrete in full tram-trucks from the low level to the bridge.

The consistency of the concrete was controlled by limiting the water used. Slump tests were made, but were not found to be very effective in indicating the workability of the concrete, which usually had to be adjusted by trial. As a result, the concrete erred on the side of wetness. Samples of the various mixes were taken daily and were tested in the laboratories of the Government School of Engineering at Rasul.

In depositing the 9-inch bedding layer, some difficulty was experienced with subsoil water-pressure, since if a large area was covered simultaneously and remained unweighted, sufficient pressure developed to crack and lift it, despite the fact that water was freely drained from the edge. In one case, an area about 40 feet square in the horizontal floor of the right pocket was lifted bodily to a height of about 8 inches. This difficulty was over-

come by limiting the area of the bedding-course which was constructed i advance of the main floor, by providing relief pipes in the bedding-course and also, in the more difficult lower levels of the pockets, by installing tube wells with 25 feet of 5-inch strainer at about 20-foot intervals and pumpin from them. In the portion of the floor which was actually damage grouting-pipes were installed at 8-foot intervals in each direction, and th cavities under the floor were subsequently grouted up after laying th upper portion of the floor, in the course of which the bedding-course re turned to within 2 inches of its correct position.

All concrete was kept wet for a full month after laying. For this pur pose an extensive system of piping was installed, fed from the pum delivery-pipes for the low-level work, and by small electric pumps in th case of the upper parts.

All reinforcement was bent locally by hand. The method of bendin the larger bars was primitive and spectacular. The bar was held i position by pegs set in concrete in the ground, the end was gripped by clamp at the end of a lever bar about 12 feet long, and this was pulle round by a team of men harnessed by ropes to the end of it. The vertica component of the pull in the harness was counterbalanced by a man wh sat on the end of the bar. The floor reinforcement was built up in positio In laying the bedding-course, chairs, manufactured locally of ½-inch bars had been built in for the dual purpose of supporting the reinforcemen and binding the bedding-course on to the main floor. In the case of th overbridge the reinforcement of the T-flanges was built up on top of th shuttering, and subsequently lowered into position. Owing to the weigh of the reinforcement, it was found necessary to use cast-iron distanc pieces to ensure the correct thickness of cover at the bottom and sides.

After erection of the reinforcement, the bedding-course was cleane by means of a water-jet, before the concrete was deposited. In the con struction of the overbridge, compressed air was similarly used for cleanin the forms.

To start with, "wriggling" the concrete into position between th reinforcement was done by hand, but at an early stage eight compressed-a vibrators were brought into use, with very beneficial effect. Some han wriggling was retained throughout, however, thus avoiding comple stoppage in the event of the breakdown of a compressor.

All previous Departmental records for the placing of concrete we broken on this work, outputs of 52,000 cubic feet per day and 900,0 cubic feet per month being obtained.

A foundation "stone", comprising a precast block bearing a bron plate, was laid at the base of pier No. 29, by Sir Herbert Emerson, G.C.I.E K.C.S.I., C.B.E., I.C.S., Governor of the Punjab, on the 10th Februar 1938, and in his honour the work was named the Emerson barrage.

*Form-Work.*—This is the first occasion in the Punjab on which co crete has been used for vertical facework on barrage construction. T

piers and abutments of previous works were built in stone or brick. In the case of the Trimmu barrage concrete was adopted in the first place owing to the absence of suitable building stone, but the method adopted was so successful that there is no doubt that it will be used again, even if stone is available. The method referred to is the use of the precast interlocking concrete shell. The type of shell used is 2 feet long by 1 foot high and 3 inches thick. It has two keys of different lengths at the back, each key being wedge-shaped and provided with notches on the upper edge and dowels on the lower, so as to interlock. The shells were precast in the station area in locally-made metal forms. The mix was 1 : 2 : 4, with $\frac{3}{4}$-inch aggregate, and the shells were lightly reinforced.

The surface was lightly rubbed after the forms had been removed, usually about 24 hours after pouring, but was otherwise untouched. Walls made with this type of construction were raised in 2-foot layers, the shells being first laid in mortar by masons, and concrete being then poured behind them; the mix was 1 : 4 : 8, with about 16 per cent. of stone "plums." Horizontal joints in the concrete were made to coincide with the centre of the shells. Similar shells, 2 feet by 4 feet, were used on the pier noses, but were rather difficult to handle. Forms of brickwork in mud mortar, with the inside plastered and wooden laths to reproduce the joints, were frequently used in these cases.

In the construction of the blockwork, brick form-work was used. This consisted of a shell of brickwork in cement mortar, 5 inches thick, with occasional headers to bind into the concrete, the shell, of course, forming part of the finished block.

The forms for the road-bridge were of wood, faced with galvanized-iron sheeting built on to a structural steel framework. At first this shuttering was supported on cribs made of sleepers, but with this form of support it was found difficult to control the settlement under load, and later recourse was had to columns of masonry in mud mortar. Six sets of shuttering were necessary to enable the road-bridge to be completed in time.

*Erection of Steelwork.*—The steelwork was manufactured at Amritsar, and was conveyed to Trimmu by rail. The sill girders and grooves were delivered in advance, and were manhandled to the site and built in as the work proceeded. The undersluice-gates were completely assembled at Amritsar, but the 60-foot weir gates came in two halves and were assembled at the site. The superstructure columns and counterweight-boxes arrived complete. The over-bridge, including the suspension-drums and counter-shafts, was assembled at Trimmu before erection.

On arrival at Trimmu, the main steelwork was unloaded on to specially-constructed skids consisting of two lines of single rail supported on wooden beams from masonry columns ,connecting, at a gentle incline, the unloading siding with a broad-gauge line, temporarily laid across the road-bridge. There were five skids on each side of the barrage, and the steelwork was

stored on these until required, in the order required for erection. The superstructure of the over-bridge was assembled at this site. The various parts were moved along the skids by winches situated at the delivery end, whence they were loaded on to roller-bearing trolleys on the road-bridge line. For the erection of the steelwork at the site, the Central Workshops designed and constructed two pairs of gantries carrying electrically-operated transverse travelling cranes. These gantries travelled on road-wheels and straddled the railway line in such a way as to permit any of the parts to be passed beneath them. With the help of these gantries the steel-work erection was a very simple matter. The superstructure columns were erected in advance with the help of a 12-ton crane, and were aligned and grouted in position. A pair of gantries was then moved into position, and the main steelwork, brought along in the order required, was picked up by the cranes and placed in position, only a few hours being required for a complete span. The two halves of the large gates were riveted together after being placed in position. A small gantry travelling on the overbridge was used for the erection of the winches, hand railings, and floor-boards. The arrangements, which were organized and operated by the Central Workshops, worked very smoothly, and the whole of the 3,500 tons of steelwork was erected in about 3 months.

*River Diversion.*—A study of the flow-records of the Chenab at Trimmu revealed that December was the most suitable time for carrying out the river diversion, and that at this time a supply of from 1,000 to 3,000 cusecs might be expected. It also showed that freshets had been experienced in all the winter months, the largest being about 20,000 cusecs in December, 30,000 in January, 50,000 in February, and 100,000 in March. In April, discharges of 200,000 to 300,000 cusecs had been experienced.

It was impossible to have the whole barrage finished by December 1938, but the right undersluice would be completed by this time and could accommodate 20,000 cusecs if necessary. A sufficient length of the barrage would be ready in succeeding months to take the largest freshets likely to occur. Accordingly it was decided to attempt the diversion at this time.

The position at the end of the 1938 flood season was that the right marginal bund and the Rangpur canal were complete up to within 2 miles of the barrage, this space having been left for the river to pass through in the past monsoon. In this area were three channels, two large creeks to the right, and the main stream passing close to the nose of the guide-banks and crossing the marginal bund alignment about 4,000 feet from the barrage. Obviously the first operation was to close the right creek, which was effected with the help of permeable barriers, comprising brushwood screens suspended from wires carried by fences of poles driven by water-jet into the river bed. The action of such a barrier is to cause the current to scour the bed beneath it, with the result that a bar is formed on the down-stream side, and the flow is reduced. In this case, three barriers reduced

the flow to such an extent that it could be closed without difficulty in October. Meanwhile, work had been started on the unfinished portion of the marginal bund, and was carried on as rapidly as possible, so that it was completed, except across the main stream, by the beginning of December. At the same time the diversion cuts were constructed and completed. The alignment of these channels is shown in Fig. 15, Plate 2. The main channels were 200 feet wide down to spring level (about 475·0), and in them cunettes 100 feet wide were constructed, partly by excavators and partly by sectional pumping, with beds sloping from 472·0 to 471·0. The branches to the weir and left pocket were 100 feet wide, with bed at spring level. Only the main channel, however, could be used for the diversion.

Some doubt existed as to the best site for the final diversion bund. A position immediately downstream of the head of the diversion cut had advantages. Wherever it was placed, between the head and tail of the cut, it would be subject to the same head of water, but the nearer it was to the head of the cut, the less would be the height of the bund relative to the river-bed. Finally it was decided to place the bund just downstream of the marginal bund, since a portion of the earthwork would thus form part of the final marginal bund at this site.

The diversion bund consisted of a stone bund, laid on brushwood mattresses, with a core wall of sandbags in wooden crates. The face of the stonework was blinded with brick ballast and covered with tarred sacking, after which an earthen bund was built on the upstream side. The mattresses consisted of rolls of brushwood about 8 inches in diameter, tied on to the nets of 8-inch mesh 8-gauge wire, 30 feet wide, and ranging in length from 40 feet to 100 feet. They were constructed on the banks, and were launched by rolling them up and into the water until they floated, after which they were unrolled, floated into position, moored to masts driven with the water-jet for the purpose, and sunk by weighting with stone. The cribs were next carried or floated into position and loaded with a 2-foot depth of sandbags, an apron of bags 10 feet wide being placed on the upstream side. The spaces between the cribs were also built up with sandbags to form piers for a foot-bridge giving access to the whole length of the work. Dumping of stone was then commenced from the bridge and also from boats. This required careful control to avoid concentration of the discharge at any point; but nevertheless several cases of settlement of the cribs occurred. With the stone above water-level, the head on the bund was about 1 foot, and a tramway was laid on the stone at this stage to accelerate the dumping. The blinding was next carried out, the tarred sacking, which had been sewn to form sheets 50 feet wide, was placed in position, and sinking was started from each side, the cribs being simultaneously filled with sandbags. The earth bund followed this process as rapidly as possible. The diversion cut was opened on the 6th December and the bund was closed on the 18th, the discharge being 1,250 cusecs and the head 4·9 feet. The labour employed was about 1,000 men, with

1,700 donkeys. The methods used were admittedly primitive, but they were effective and the cost of the diversion at Trimmu (about 70,000 *rupees*) was lower than for any of the Sutlej Valley project headworks.

*River Development.*

After the closing of the river, the discharge fell slowly to about 1,000 cusecs, notwithstanding which the diversion cut developed until it was passing the discharge with a slope of 1 in 5,000, which is about the normal river slope, by the end of January.

In December it had been realized that, owing to delays in the construction of the over-bridge, it would not be possible to open more than fifteen bays of the weir until February, and whilst this was ample for any freshet which was likely to occur during that period, the water-way leading to it was inadequate, as the central leading cut led into the left portion of the weir and would not be available for use until that was opened. Accordingly an additional cut was made in the right half and, at the same time, a strong cross-bund was made, passing through bay 22, to segregate the left half, should it be necessary to open the right half.

After the completion of a work of this description, there is always a very anxious period between the time of the river-diversion and the passing of high discharges in the ensuing flood-season. The diversion cuts must, of course, be of relatively small capacity, and the river is depended upon to develop them sufficiently to provide an adequate waterway for floods, through the relatively high land existing between the barrage and the old river bed on the upstream and downstream sides. Given a gradual slow rise of the river, this waterway will be developed with a small afflux and without danger; but should the river rise rapidly to high flood-level before the waterway has developed, the upstream levels may become so high as to endanger the marginal bunds and guide-banks.

With a given discharge a single cut will develop only to a certain extent, namely, sufficiently to provide the requisite waterway with the normal river-slope. It is possible, however, by providing a number of cuts, isolating them from each other by cross-bunds, and utilizing the barrage-gates to divert the supply from one to another, to develop each cut in turn, and thus to multiply the cross-sectional area of the developed waterway by the number of the cuts so provided.

The diversion cuts at Trimmu were not very well planned in this respect. To obtain the best results from regulation development, it is advisable that the cuts should be independent of each other through the whole area which it is desired to develop. They should also run parallel to each other and be spaced evenly, so that as large an area as possible may be developed before one cut breaches into another.

The cross-bunds which are required to separate the areas served by adjacent cuts in the vicinity of the barrage present difficulty in construction. They have to cross the block-and-stone area, and whilst the stone

apron can be made reasonably watertight by sand-grouting, it is difficult to make the stone under the blocks impervious, and undesirable to attempt that in the filter-area. The only effective way of enabling the cross bunds to withstand any considerable head in these areas is to provide a curtain-wall or pile-line beneath them. This should extend from the end of the impervious floor through the filter and blocks, and well into the stone area which should, of course, be sand-grouted under the bund. The provision of such cut-offs at suitable intervals should be an essential part of future barrage design. Apart from river development, they will be useful if it is desired in the future to isolate and unwater any portion of the barrage floor for maintenance purposes.

These principles were realized at the time when it was decided to provide the additional cut in the right half of the weir, and this cut was consequently carried through to the river, both upstream and downstream, as shown on Fig. 15, Plate 2. The cross-bund at bay 22 was also provided with a curtain-wall in the block-and-filter area.

The development of the second cut by regulation was not, however, possible, because shortly after it was completed, a freshet of about 20,000 cusecs was experienced, which was passed through the right half of the weir and pocket without difficulty, and developed both cuts in a way that could not have been done by regulation with a low discharge.

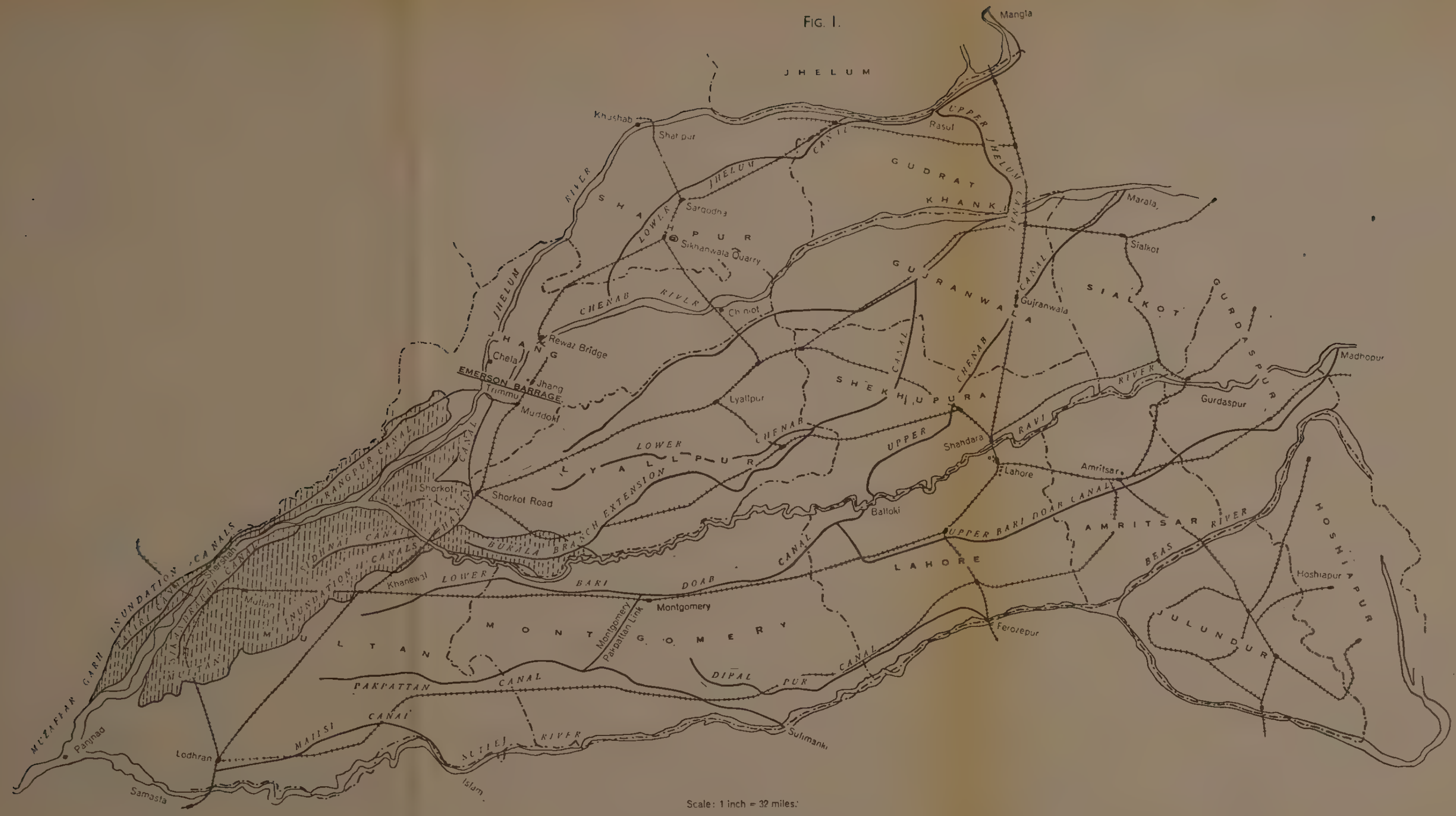
On the 3rd March a record winter flood was experienced, the discharge of which is not known with accuracy, but was probably between 125,000 and 150,000 cusecs. The combined discharges at Rasul and Khanki were 250,000 cusecs, and that at Panjnad was 72,000 cusecs. At this time the left half of the weir and pocket were completed, but had not been opened. The gates had not been erected in the Haveli canal regulator, but temporary walls had been built across the orifices to provide against the contingency of its being necessary to admit water into the pocket. With the right half of the weir and pocket open, the water-level upstream rose to 492·3, in comparison with the old maximum flood-level of 490·5 and the designed maximum of 493·5. The flood was being passed with perfect safety, but in the interests of the inhabitants of the 25 square miles of pond area, who had not yet been removed, and were obviously in trouble, it was decided to open the left half. This lead to a minor catastrophe. Owing to the rapid development of the upstream cuts while the weir-area was filling up, unexpectedly high levels were obtained on the left flank. The temporary walls in the regulator failed, and water entered the canal, where many works, including the two canal silt-ejectors, were still incomplete. Fortunately, the railway embankment, 2 miles down the canal, had not been removed, and this limited the length of canal which was inundated, but water emerging from the silt-ejector gaps partially flooded the station area. The water interrupted telephonic communications, and there was consequently uncertainty for some time whether the river was still rising and what the extent of the flooding would be. Most of the population of the

station therefore spent an uncomfortable night on the roofs of their houses or in railway-wagons. Actually the river started to fall shortly after the left side of the barrage had been opened, and it was subsequently found that the damage done in the station-area and to canal works was not serious. The time lost by this set-back was subsequently made up, and the headworks were duly opened by Sir Henry Craik, Bart., K.C.S.I., I.C.S., Sir Herbert Emerson's successor, on the appointed date, namely, the 2nd April, 1939.

The Paper is accompanied by nineteen sheets of drawings, from which Plates I and II, and the Figures in the text, have been prepared, and by nine photographs.

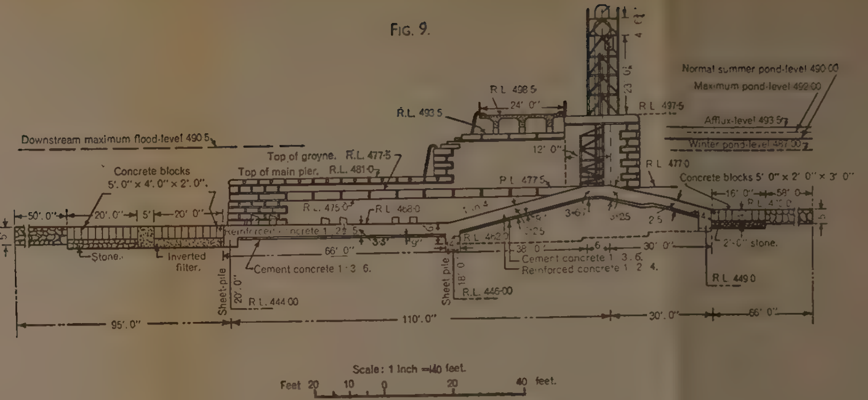
# THE EMERSON BARRAGE.

Fig. 1.



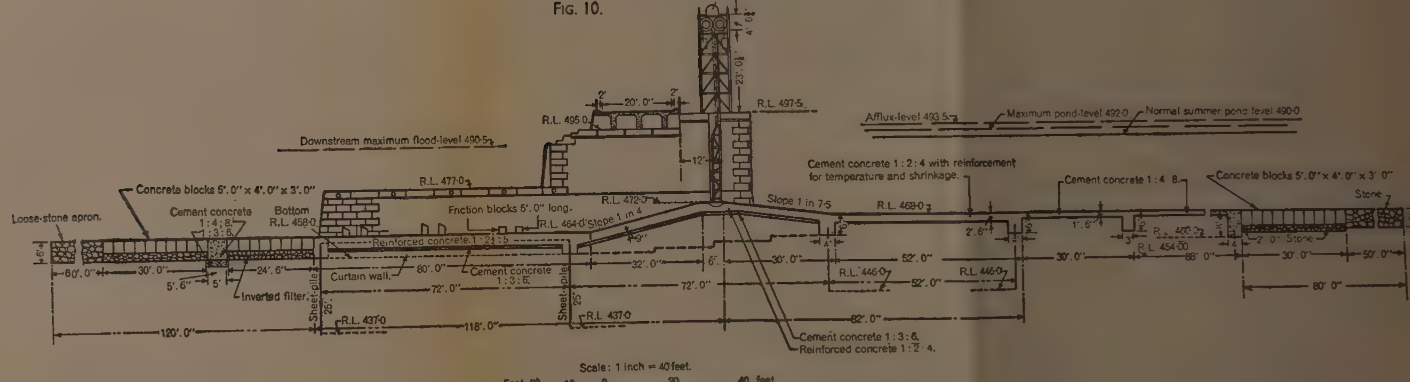

GENERAL PLAN OF THE HAVELI PROJECT.

Fig. 9.




SECTION THROUGH WEIR BAY.

Fig. 10.



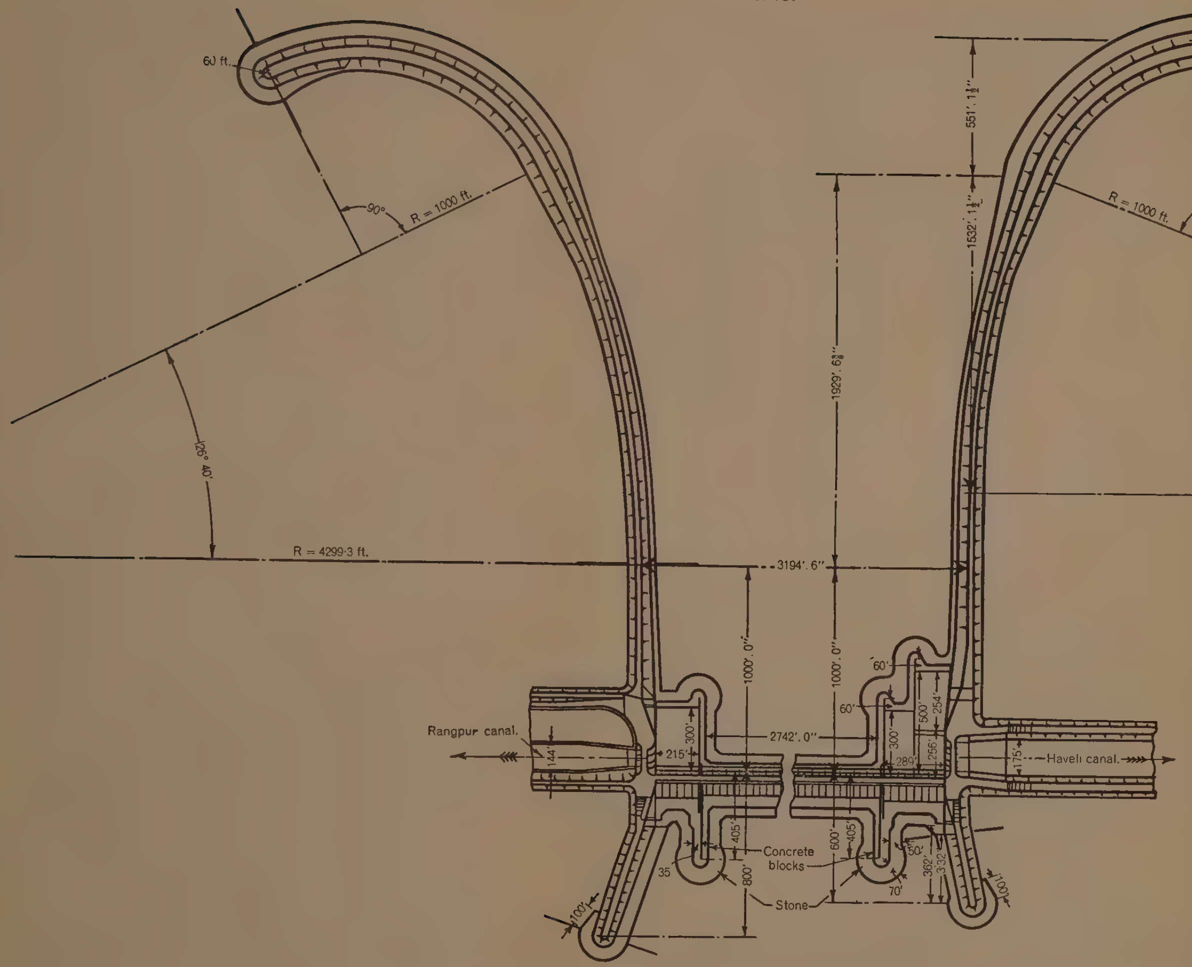

SECTION THROUGH UNDERSLUICES.

F. F. HAIGH.

The Institution of Civil Engineers. Journal. December, 1941.

William Clowes & Sons, Limited: London.

Fig. 13.

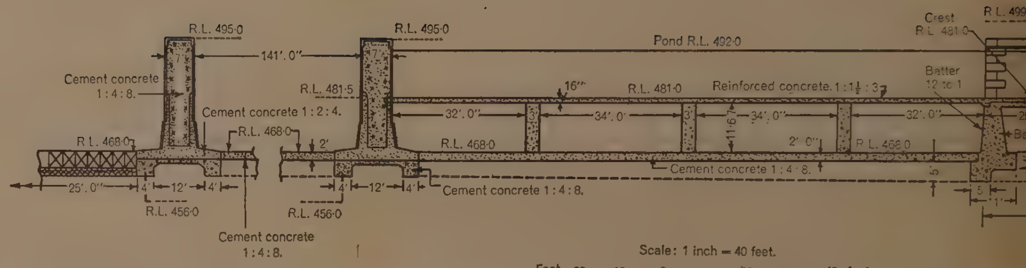



Scale: 1 inch = 800 feet.

Feet 400 200 0 400 800 1,200 feet.

GENERAL PLAN OF BARRAGE.

Fig. 14.




Scale: 1 inch = 40 feet.

Feet 20 10 0 20 40 feet.

SECTION THROUGH THE HAVELI MAIN-LINE REGULATOR AND POCKET

William Clowes & Sons, Limited: London.

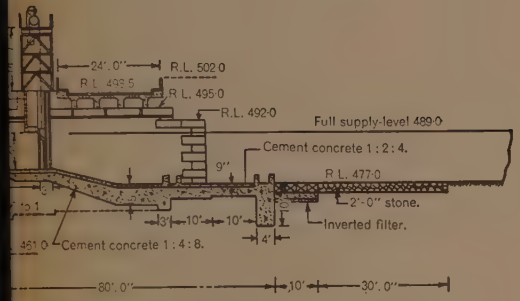



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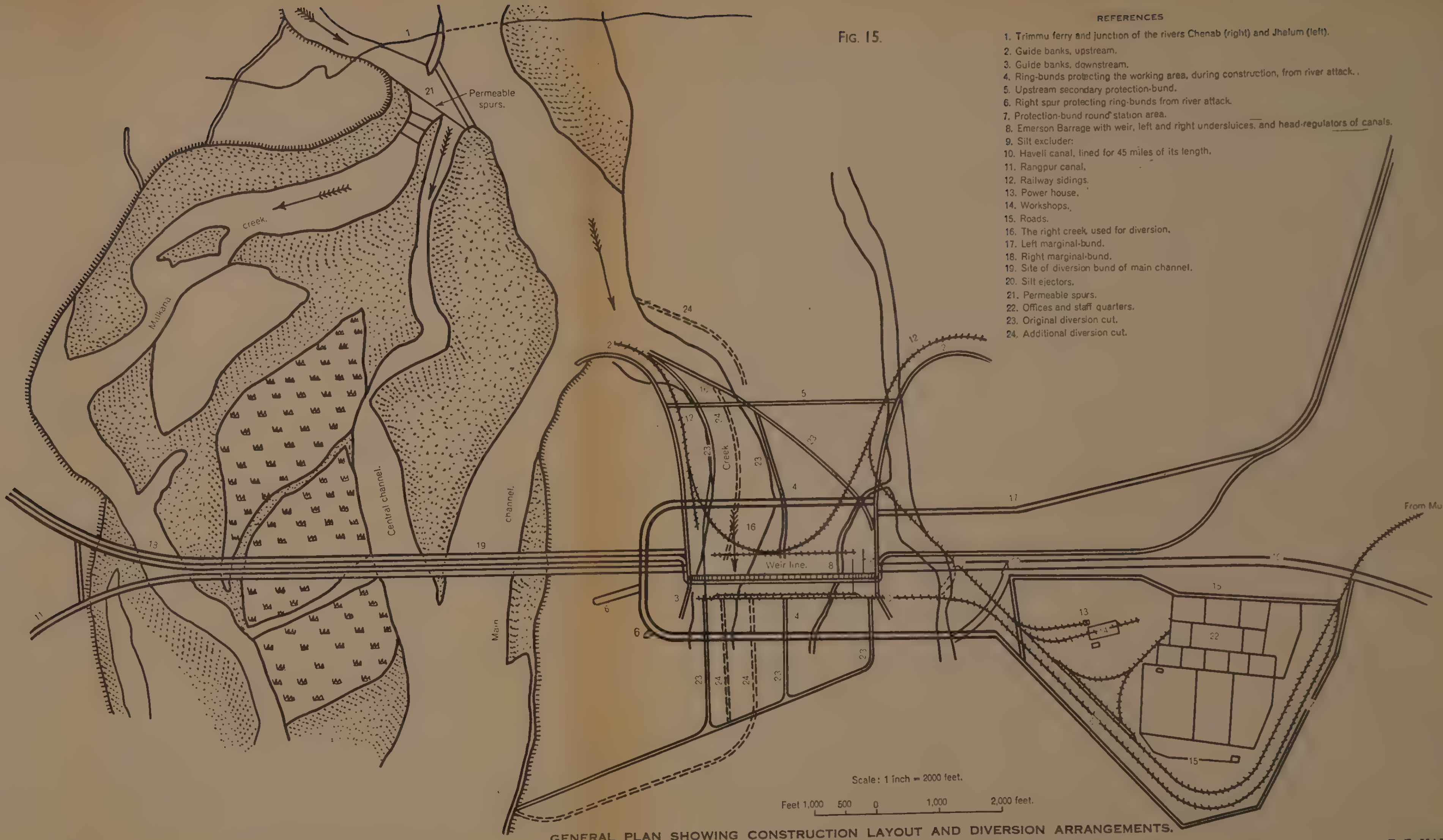
FIG. 15.




REFERENCES

1. Trimmu ferry and junction of the rivers Chenab (right) and Jhelum (left).
2. Guide banks, upstream.
3. Guide banks, downstream.
4. Ring-bunds protecting the working area, during construction, from river attack.
5. Upstream secondary protection-bund.
6. Right spur protecting ring-bunds from river attack.
7. Protection-bund round station area.
8. Emerson Barrage with weir, left and right undersluices, and head-regulators of canals.
9. Silt excluder.
10. Haveli canal, lined for 45 miles of its length.
11. Rangpur canal.
12. Railway sidings.
13. Power house.
14. Workshops.
15. Roads.
16. The right creek used for diversion.
17. Left marginal-bund.
18. Right marginal-bund.
19. Site of diversion bund of main channel.
20. Silt ejectors.
21. Permeable spurs.
22. Offices and staff quarters.
23. Original diversion cut.
24. Additional diversion cut.

Scale: 1 inch = 2000 feet.

Feet 1,000 500 0 1,000 2,000 feet.

GENERAL PLAN SHOWING CONSTRUCTION LAYOUT AND DIVERSION ARRANGEMENTS.

F. F. HAIGH.

Paper No. 5262.

# "Steel Pedestals for Heavy Columns."

By Vernon Ferdinand Bartlett, B.Sc. (Eng.), M. Inst. C.E., and John Young Danks, B.Sc. (Eng.), Assoc. M. Inst. C.E.

*(Ordered by the Council to be published with written discussion.)* [1]

The heavy loadings on column bases, which occur in power-station construction, usually require the use of substantial steel grillages to distribute the load over an adequate area of the foundation concrete.

In the Battersea "A" station of the London Power Company the column loadings were unusually high. Especially was this the case in the chimney tower structure, where the column loads ranged from 1,400 tons to 1,900 tons. Here double-tier grillages were employed, comprising a top tier of three plate girders, 4 feet in depth, and a lower tier of heavy rolled steel joists. Both tiers were fitted with web stiffeners of heavy angle section, and stout diaphragm-plates, riveted to the stiffeners, served to tie together the girders forming the top tier. A steel slab base, 8 inches thick in the maximum case, was interposed between the foot of the column and the machined bearing plate, which was riveted to the top flange of the girders.

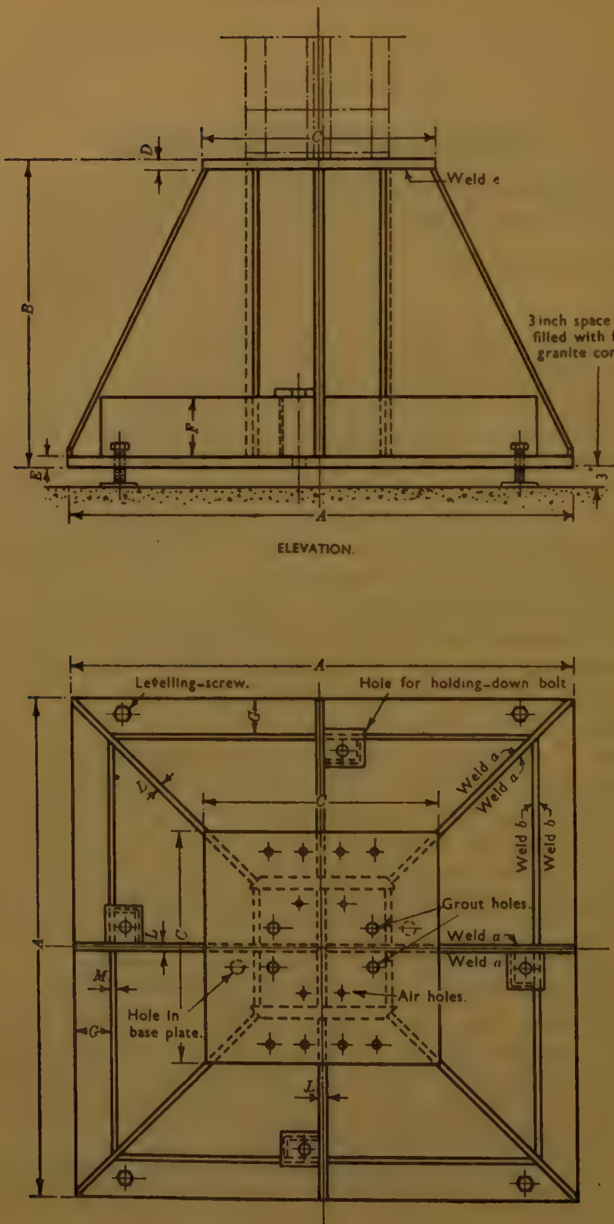
The quantity of steelwork involved in such grillages was considerable and difficulty was experienced on site in setting, levelling, and filling the grillages satisfactorily. It was necessary to concrete the lower tier of grillage beams before accurately setting the top tier. Furthermore, the column had to be set up and plumbed before grouting in the top tier. These operations usually resulted in the loss of steel-to-steel contact between top and bottom tiers over a large area, and it was a matter of considerable difficulty to ensure solid filling of such spaces.

When the recent extension to the Battersea power-station was undertaken, consideration was given to the possibility of replacing grillages by some form of pedestal, which would disperse the load in a more direct manner, eliminating, as far as possible, the bending stresses involved in a grillage, and which would also have the advantage of ease in setting accurately to line and level on the foundation.

Whilst the type of pedestal adopted involves no new principle of design, its use was found to be fully justified on both practical and economical

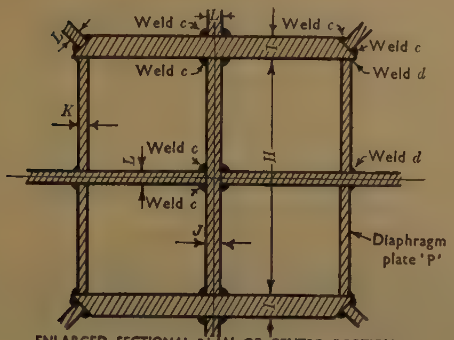
[1] Correspondence on this Paper can be accepted until the 15th April 1942, and will be published in the Institution Journal for October 1942.—Sec. Inst. C.E.

*Figs. 1 (a) and (b).*




DETAILS OF PEDESTAL.

*Figs. 1* (*c*).




ENLARGED SECTIONAL PLAN OF CENTRE PORTION

TABLE OF DIMENSIONS.

| Load: tons. | 1780 | 1280 | 930 | 770 | 610 | 460 | 340 |
|---|---|---|---|---|---|---|---|
| Weight: cwt. | 105 | 76 | 49 | 37 | 26 | 18 | 15 |
| *A* | 7′ 0″ | 6′ 0″ | 5′ 0″ | 4′ 6″ | 4′ 0″ | 3′ 6″ | 3′ 0″ |
| *B* | 4′ 4″ | 4′ 3″ | 3′ 9″ | 3′ 0″ | 2′ 6″ | 2′ 0″ | 2′ 0″ |
| *C* | 3′ 3″ | 3′ 0″ | 3′ 0″ | 2′ 9″ | 2′ 8″ | 2′ 6″ | 2′ 6″ |
| *D* | 2″ | 1¾″ | 1½″ | 1½″ | 1½″ | 1¼″ | 1¼″ |
| *E* | 2″ | 1¾″ | 1½″ | 1½″ | 1¼″ | 1″ | 1″ |
| *F* | 10″ | 10″ | 9″ | 8″ | 7″ | 6″ | 5″ |
| *G* | 6″ | 5½″ | 4½″ | 4″ | 3¾″ | 3½″ | 3″ |
| *H* | 1′ 8″ | 1′ 8″ | 1′ 8″ | 1′ 6″ | 1′ 5″ | 1′ 4½″ | 1′ 4½″ |
| *I* | 2″ | 1¾″ | 1½″ | 1½″ | 1½″ | 1¼″ | 1¼″ |
| *J* | 1½″ | 1¼″ | 1¼″ | 1¼″ | 1¼″ | 1¼″ | 1¼″ |
| *K* | 1″ | 1″ | ¾″ | ¾″ | ⅝″ | ⅝″ | ⅝″ |
| *L* | 1¼″ | 1″ | ¾″ | ¾″ | ⅝″ | ⅝″ | ⅝″ |
| *M* | 1¼″ | 1″ | ¾″ | ¾″ | ⅝″ | ⅝″ | ⅝″ |
| *a* | 9/16″ | 7/16″ | 5/16″ | 5/16″ | 5/16″ | 5/16″ | ¼″ |
| *b* | 9/16″ | 7/16″ | 5/16″ | 5/16″ | 5/16″ | 5/16″ | ¼″ |
| *c* | ½″ | ⅜″ | ¼″ | ¼″ | ¼″ | ¼″ | ¼″ |
| *d* | ¼″ | ¼″ | ¼″ | ¼″ | ¼″ | ¼″ | ¼″ |
| *e* | ¼″ | ¼″ | ¼″ | ¼″ | ¼″ | ¼″ | ¼″ |

grounds; it is therefore felt that the details may be of interest as offering an alternative to the use of a grillage, particularly in the case of heavy column loading, or where grillage space is limited.

## Design.

*Figs. 1* indicate the form finally adopted. It consists essentially of a top bearing plate, connected by means of a core of H-section with radiating ribs, to a base plate, the latter being of sufficient area to ensure a pressure not exceeding 620 lb. per square inch. This pressure was specified for reinforced concrete of proportion: 4 cubic feet of ¾-inch crushed shingle to 2 cubic feet of sand and 1 cwt. of cement.

Fabrication by welding appeared to be desirable, and the use of this method was agreed to by the special Buildings Department of the London County Council, on condition that the welding be carried out by the electric arc process with heavily-coated electrodes, and supervised by an approved inspector.

The basis of the weld design was B.S.A. Specification No. 538—1934, but with the following reduced working stresses in the welds:—

| | |
|---|---|
| Compression . . . . . . | 6·5 tons per square inch. |
| Tension . . . . . . | 5 tons per square inch. |
| Shear . . . . . . . . | 5·5 and 4·5 tons per square inch in end and side fillets respectively. |

A range of seven pedestals was designed for single-shaft columns carrying loads of between 1,780 tons and 340 tons; these were square in plan, as shown in *Figs. 1*. Details of their construction are given in the accompanying Table of dimensions (*Figs. 1* (c)).

Two further sizes of pedestal, rectangular in plan, were designed for double-shaft columns supporting loads of 1,140 and 1,440 tons respectively. This type is illustrated in *Fig. 2*.

In calculating the sections required for the various members the permissible stresses used were those given by B.S.A. and L.C.C. specifications for steel grillages buried in concrete—namely, 50 per cent. over the usual working stresses for steel structures. An exception was made, however, in arriving at the bearing area provided by the core and ribs immediately below the cap plate. Here the stress was limited to the usual 12 tons per square inch; the effective area being assessed by assuming a dispersion angle of 45 degrees through the thickness of the cap plate from the boundary of the column cross-section.

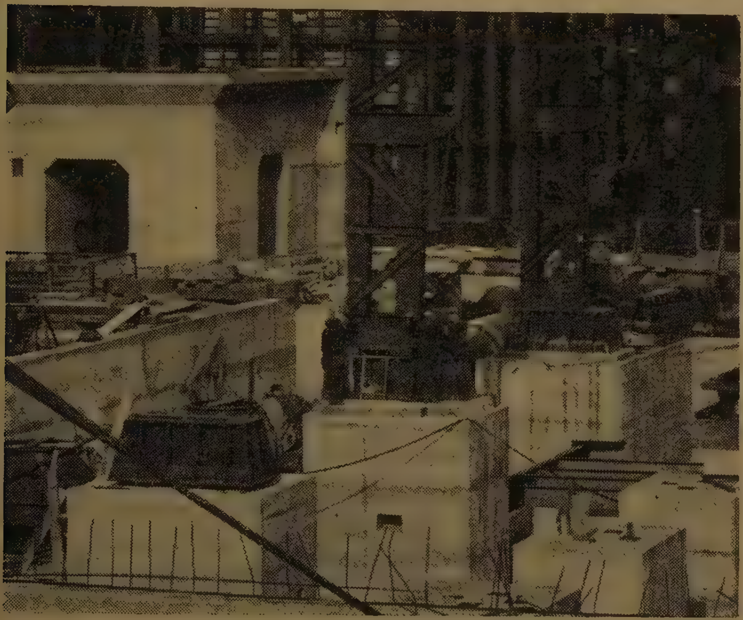
It will be noted, in this connexion, that the column section should not be treated as a whole, but should be regarded as composed of separate web and flange areas, each of these areas being treated separately in order that the cross-sectional area provided in the separate parts of the core may b

proportionate to the loading imposed by the various sections of the column above.

The diaphragm plates P were not included in the effective area, these being regarded as stiffeners, in view of the fact that their attachment was by means of light welds on the outside only.

The columns were of H-section, composed of plates and angles, the dimensions over the bottom section of the largest being 27½ inches by 24 inches. This section comprised a web plate 20 inches by 1½ inch, four

*Fig. 2.*



Rectangular Pedestals.

angles 8 inches by 8 inches by 1 inch, and two flange-plates 24 inches by 3¾ inches.

Having regard to the importance of good bearing above and below the cap plate, the latter was machined on both sides; the core and rib plates were machined on all vertical and horizontal bearing edges. Machining was not considered necessary, however, in the case of the base plate, where the bearing stress on combined core and rib cross-section did not exceed 3·5 tons per square inch. The average compressive stress over a cross-section at mid-height of the pedestals was generally between 4·5 tons and 5 tons per square inch.

In ascertaining the stresses in the individual ribs and their weld fillets,

the loads were obtained from consideration of the base area supported b[y] each rib. *Fig. 3* shows the forces acting on one of the $1\frac{1}{4}$-inch thic[k] diagonal ribs in the largest pedestal, where the compressive stress on cross-section normal to the line of thrust was $\frac{301}{32} \times 1{\cdot}25 = 7{\cdot}53$ tons pe[r] square inch.

The vertical and horizontal welds attaching this rib were respectivel[y] $\frac{1}{2}$-inch and $\frac{9}{16}$-inch fillets, the shear stresses being 6 tons and 6·5 tons pe[r] square inch of throat-area.

The thickness of the base plate was determined by considering a stri[p]

*Fig. 3.*


Diagram of Forces Acting on a Rib.

in the middle of the trapezoidal panels, bounded by the main ribs and th[e] outer stiffening rib. This strip was assumed to carry a uniformly di[s]tributed loading of intensity equal to one-half of the bearing pressu[re] under the base plate. The plates varied in thickness from 2 inches in th[e] largest to 1 inch in the smallest size pedestals.

In considering the strength of the peripheral stiffener plate, a portio[n] of the base plate adjacent to it was included to form a symmetrical invert[ed] T-section.

## Shop Work.

The method of assembly adopted in the construction of the pedesta[ls] was as follows :—

(1) The H-section core was welded, and the ends were milled square to length.
(2) The core was welded to the base plate.
(3) The eight main ribs were then welded to the base and to the core section, the pedestal being mounted horizontally on a mandrel for easy rotation during the latter operation.
(4) Finally, the cap plate, peripheral stiffeners, diaphragm plates, and holding-down-bolt boxes were welded on.

Some trouble was experienced in welding the ribs to the core at points where four welds occurred close together. It was found, in running the fourth weld, that cracking occurred in one or other of the adjacent fillets. This trouble was overcome by departing from the usual method of building up the weld by making a number of longitudinal runs. The process adopted consisted of building the fillet to its full cross-section, by transverse movements of the electrode while working slowly in a longitudinal direction.

In several cases it was found that the cap plate had warped slightly after welding. It was therefore decided to postpone the machining of the top surface of the cap plate until after the final assembly of the pedestal.

## Site Erection.

Ease of levelling was obtained by providing four levelling-screws, placed near the corners of the base, and these were made of sufficient length to leave a space of 3 inches between the underside of the base plate and the concrete formation. A space of only 1 inch between base plate and concrete is very often adopted, but there is reason to doubt whether bearing is obtained over the full base area by the process of filling such a space through grout holes in the base plate.

The process of setting the pedestals was carried out in the following sequence :—

(1) The concrete foundation block was completed to within 3 inches of the underside of the base of the pedestal.
(2) The pedestal was lifted into position and set approximately to line and level. The heads of holding-down bolts were then concreted in.
(3) Final accurate adjustment of the pedestal was completed by means of levelling-screws, bearing on small steel plates laid on the concrete and working against the holding-down bolts.
(4) The space between concrete and pedestal was filled.

The adoption of a 3-inch space permitted the use of fine granite concrete filling; this was rammed by means of special hand tools, which were worked simultaneously from opposite sides of the base, to ensure

solid packing over the entire area, the holding-down bolts preventing uplift.

After setting the pedestal, the centre pockets were grouted up, grouting- and air-escape holes being provided in the cap plate for this purpose. The surrounding concrete was later brought up to floor level, steel hoop reinforcement being incorporated to contain the pedestal filling.

The type of pedestal described above proved to be entirely successful in the Battersea extensions, and was shown to possess the following advantages :—

(*a*) *Cost and Weight.* Whilst the welding process increases the price per ton of manufacture, the saving in steel in comparison with the ordinary grillage is large, amounting in the case of the heavier types to nearly 50 per cent. The ultimate cost should show a useful saving, especially when the simpler site erection is taken into consideration.

(*b*) *Site Erection.* With the aid of the steel-erection crane, the pedestals were planted in position in a very few minutes. High accuracy was possible in the final adjustment by means of the levelling-screws and an engineer's spirit-level.

The columns were not placed until the concrete packing had set, but in every case they proved to be truly vertical, and the erection of the adjoining steelwork followed without delay.

The Authors are indebted to Sir Leonard Pearce, C.B.E., D.Sc., M. Inst. C.E., Engineer-in-Chief to the London Power Company, for permission to publish this Paper.

The pedestals were fabricated by Messrs. Sir William Arrol & Co., Ltd., contractors for the main steelwork. Mr. P. J. Hunter acted as steelwork inspector at the contractor's works on behalf of the London Power Co., Ltd.

The Paper is accompanied by one sheet of drawings and by two photographs, from which the Figures in the text have been prepared.

Paper No. 5274.

# "The Ultimate Bearing Pressure of Rectangular Footings."

By Hugh Quintin Golder, M.Eng., Assoc. M. Inst. C.E.

(*Ordered by the Council to be printed with written discussion.*) [1]

TABLE OF CONTENTS.



## Introduction.

In foundation design the stability of a footing must be considered from two different points of view, since there are two ways in which failure can take place, namely, by excessive settlement of the footing due to consolidation or deformation of the underlying soil strata, or by shear failure of the soil. The allowable bearing pressure will be the smaller of two values, namely: (1) the pressure which causes settlements which do not exceed a certain specified value; (2) the pressure which gives a chosen factor of safety against shear failure.

In this Paper only the question of shear failure is considered, and the term "ultimate bearing pressure" refers to the pressure which causes shear failure.

The design of a foundation against shear failure may be based upon bearing tests on the soil, or upon theoretical stability analysis using values of soil characteristics obtained from laboratory tests.

Bearing tests are usually made on small and approximately square areas, but the results are applied to the design of much larger footings, often rectangular in shape. Before the extrapolation from the small test-areas to the larger footings can be made with confidence, the influence of the size and the shape of a footing upon the ultimate bearing pressure should be known. Tests on full-size footings, however, are very expensive,

[1] Correspondence on this Paper can be accepted until the 15th April 1942, and will be published in the Institution Journal for October 1942.—Sec. Inst. C.E.

because of the large loads required to cause failure and, generally, recourse must be had to tests on small footings of various sizes and shapes. Such tests will give an indication of the effect of these factors upon the bearing pressure, although large-scale tests are always desirable when circumstances permit.

If the problem is approached by means of a theoretical stability-analysis, it will be found that various simplifying assumptions must be made; and, in general, only the two-dimensional case of an infinitely long strip footing has been attempted. Many methods of analysis have been developed in very different ways, but all indicate that the ultimate bearing pressure is directly proportional to the width of the footing on a purely frictional soil, and independent of the width on a purely cohesive soil.

Before these theoretical methods can be used with confidence, they must be checked by practical tests. Tests on small footings can be used to determine the effects of the different variables, and to compare the theoretical and the actual bearing pressures. The purpose of this Paper is to review the tests which have been made on sand by various investigators, and to describe further tests which were found to be necessary to clear up a particular point, namely, the effect of the length of a rectangular footing of given width upon the ultimate bearing pressure. The values of bearing pressure obtained in these tests are compared with those calculated by different methods. The results of a few tests on clay are also included and their correspondence with the theories is examined.

The difference in the behaviour of footings on sand and on clay requires that the two cases be treated separately.

## Previous Bearing Tests on Sand.

Several investigators have made tests on small footings on sand.

Fellenius [1] carried out tests on sand, using rectangular and elliptical footings ranging up to 30 centimetres in width and of lengths up to 59 centimetres. His results were tabulated, and the conclusion drawn was that "on a purely frictional soil the maximum possible load per unit area of a rectangular or elliptical area is determined by the smallest dimension of the area and is proportional to this dimension."

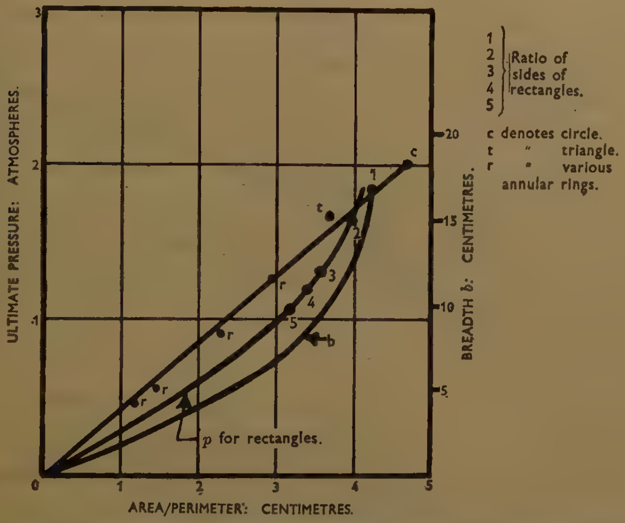
Kögler [2] also carried out tests on sand, using circular footings ranging up to 16 centimetres in diameter. His conclusion was that ultimate pressure is proportional to the diameter of the footing. He also investigated the effect of the shape of the footing on the bearing pressure, using a series of footings of equal area, but of different shapes, and observed that the ultimate pressure increased as the ratio of the area to the perimeter of the footing increased. His conclusion was that for

[1] The references are to the bibliography in the Appendix.

footings of equal area the ultimate pressure increases as the footing becomes more compact.

In another section of his book (p. 179), Kögler gives the ultimate pressure for a square footing as twice that of a long strip footing of equal width; but in spite of the fact that this statement contradicts the conclusion of Fellenius, whose results, however, he appears to accept, he gives no supporting evidence. This statement of Kögler's introduced confusion into the subject, which it is important to dispel. If Kögler's own results are examined they will be found to substantiate Fellenius' conclusion rather than his own statement that a square is twice as strong as a strip of the

*Fig. 1.*



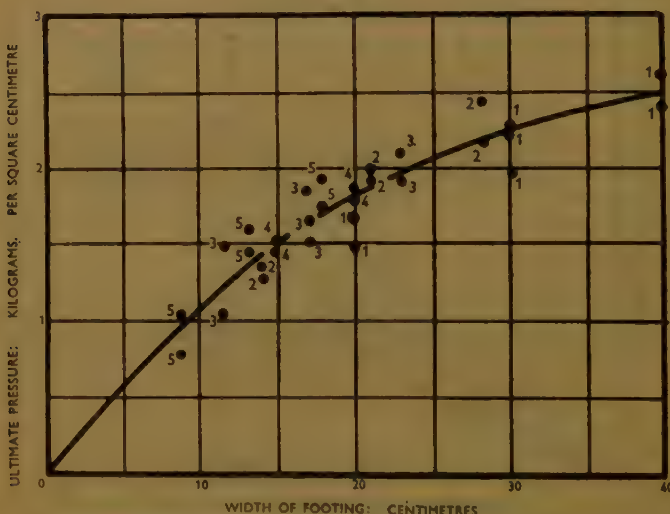

Kögler's Results: Relation between Ultimate Pressure and Shape of Footing.

same width. Kögler's results on variously shaped footings of equal area are given in *Fig. 1*. It will be seen that the rectangles do not lie on a straight line, but rather on a curve of the type marked $p$. Recently Meischeider[3] carried out tests on rectangular footings of three different areas, and obtained similar curves for each series of rectangles of constant area. The remarks which follow apply equally to the results obtained by both Meischeider and Kögler.

For a rectangle of constant area but varying shape, the breadth $b$ can be calculated and plotted against the area : perimeter ratio, curve $b$, *Fig. 1*, giving a similar general shape to curve $p$. Now if the ultimate pressure were directly proportional to $b$, but independent of the length $l$ of the footing, curves $p$ and $b$ would be expected to coincide. The difference between these two curves can be explained either by assuming that the

ultimate pressure increases slightly with $l$, or that $p$ is not directly proportional to $b$, but that the rate of increase of $p$ decreases as $b$ increases. Either of these explanations would also account for the fact that if $p$ is plotted against $b$, a curve of the type shown in *Fig. 2* is obtained. Which explanation is correct cannot be determined from Kögler's test results, but Meischeider's results, since they included three different areas, afford some indication. In *Fig. 2* Meischeider's results are plotted against the width of the footing. The figure beside each test result represents the ratio of the sides of the particular footing $n$. Although a certain scatter is evident between the individual points, the $p$–$b$ graph for any particular value of

*Fig. 2.*




MEISCHEIDER'S RESULTS: EFFECT OF WIDTH OF FOOTING.

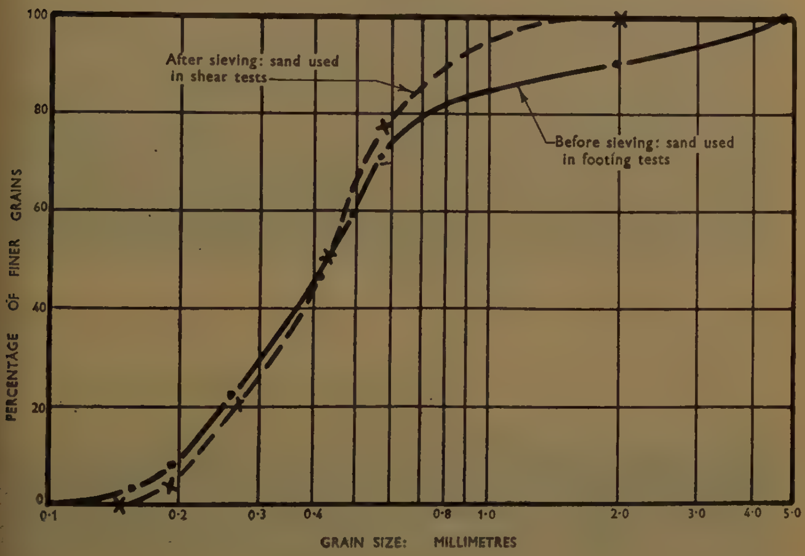
$n$ is noticeably curved. The results for the square footings seem to lie rather lower than those for the rectangles, between which there is little difference. Meischeider stated that he obtained no clearly-defined value for the ultimate pressure for the squares, so that it is doubtful whether or not this difference means that an increase of pressure occurs with length. Therefore, it is important to determine whether the ultimate pressure increases with the length of the footing.

## Loading Tests on Footings of Different Lengths on Sand.

In order to decide the effect of the length of the footing upon the ultimate pressure, a series of tests were carried out with footings of various lengths, but of equal width, on sand.

The sand used was a clean dry river sand of $\frac{3}{16}$ inch and less. The grading curve is given in *Fig. 3*. The specific gravity of the particles was 2·67. A stout wooden box, 3 feet 6 inches by 3 feet 6 inches and 14 inches deep, was filled with the sand, which was well tamped with a 3-inch square steel plate attached to an electric hammer. The voids-ratio of the sand was measured by placing two cylindrical metal vessels, open at the top, upright in the sand and about half-way up the box. After the test these containers were carefully dug out, the sand was struck off level with the top, and they were weighed. The value of the voids-ratio was found not to vary to any great extent from test to test so long as a standard technique was adopted for the compaction. The variation in weight per cubic foot

*Fig. 3.*




GRADING CURVE FOR SAND.

was from 108 lb to 112 lb., with a mean value of 110·5 lb., corresponding to a voids-ratio of 0·51.

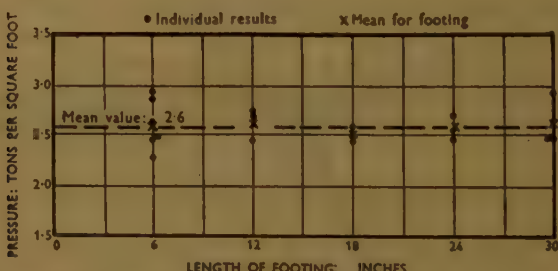
The footings used were five 6-inch rolled-steel channel sections, respectively 6 inches, 12 inches, 18 inches, 24 inches, and 30 inches in length, placed on their backs and filled with concrete; a few tests were also made on footings 3 inches wide, made in the same way from 3-inch channel section.

The footing to be tested was placed in the centre of the box on the surface of the sand, which had been carefully levelled. The load was then applied by a hydraulic jack through a proving-ring by which the load was measured. The settlement was recorded by means of a vertical

scale attached to the footing. The loading was continuous, and readings of load and deflexion were taken at intervals during the test, which was continued after the point of maximum load had been reached. Between each test the sand was turned over with a spade, compacted twice with the electric hammer, and then carefully levelled off. The rise in the sand-surface was recorded after each test by spot levels and photographs.

The results of the tests are given in Table I, and are illustrated graphically in *Fig. 4*, in which the ultimate bearing pressure is plotted against the length of the footing. It is clear that, despite a certain scatter of the individual results, there is no tendency for the ultimate pressure to increase with length. The mean pressure for all of the tests was 2·6 tons per square

*Fig. 4.*




VARIATION OF BEARING PRESSURE WITH LENGTH OF FOOTING: 6-INCH FOOTINGS.

foot, and the maximum deviation of any single test from this value was 13 per cent.

The test apparatus used was not found suitable for the small loads required for the 3-inch wide footings, and only three tests were carried out. The mean ultimate pressure for these tests was 1·4 ton per square foot, which is 8 per cent. higher than half the mean value for the 6-inch footings, thus agreeing with Meischeider's results in indicating a curve for the $p$–$b$ graph.

TABLE I.

| Test No. | 1. | 2. | 3. | 4. | 5. | 6. | 7. | 8. | 9. |
|---|---|---|---|---|---|---|---|---|---|
| Size of footing: inches . . . | 6×6 | 6×6 | 6×6 | 30×6 | 30×6 | 24×6 | 24×6 | 18×6 | 18×6 |
| Ratio of sides . . | 1 | 1 | 1 | 5 | 5 | 4 | 4 | 3 | 3 |
| Ultimate pressure: tons per square foot . . . . | 2·61 | 2·45 | 2·86 | 2·46 | 2·46 | 2·42 | 2·68 | 2·41 | 2·58 |

*Fig. 5.*

FAILURE SURFACES IN TEST NO. 16.

*Fig. 6.*

FAILURE SURFACES IN TEST NO. 9.

*Fig. 7.*

FAILURE SURFACES IN TEST NO. 4.

| Test No. | 10. | 11. | 12. | 13. | 14. | 15. | 16. | 17. |
|---|---|---|---|---|---|---|---|---|
| Size of footing : inches | 12×6 | 12×6 | 6×6 | 6×6 | 24×6 | 12×6 | 6×6 | 30×6 |
| Ratio of sides | 2 | 2 | 1 | 1 | 4 | 2 | 1 | 5 |
| Ultimate pressure : tons per square foot | 2·73 | 2·67 | 2·25 | 2·47 | 2·53 | 2·44 | 2·91 | 2·93 |

During the tests the sand rose all round the footing, and the outline of the surface of failure was clearly marked on the surface of the sand. Typical photographs are given in *Figs. 5, 6*, and *7* for the 6-inch by 6-inch footing (test no. 16), the 18-inch by 6-inch footing (test no. 9), and the 30-inch by 6-inch footing (test no. 4) respectively. In *Fig 5* the separate slip surfaces opposite the sides of the footing were the first to form. As the footing was pushed farther into the sand the continuous slip-surface appeared at a greater distance from the footing than the separate slip-surfaces. It is interesting to note that Meischeider obtained no failure surfaces with square footings, or at the ends of the long footings.

These tests indicate that for footings of the size used, on sand, the ultimate bearing pressure is independent of the length of the footing. This fact having been established, it can be concluded that the curvature of the $p$–$b$ graphs for Kögler's and Meischeider's results is not due to the effect of differences in length of footings, and that therefore the pressure is not directly proportional to the breadth, but the rate of increase of pressure decreases as the breadth increases. It is suggested that this decline in the rate of increase of pressure may be connected with the decrease in the angle of internal friction of a dense sand with increase in pressure, referred to below. The fact that Fellenius obtained a straight-line relation between $p$ and $b$ may possibly be explained by the fact that his were the only tests in which it was not stated that the sand was compacted, and possibly it was in a loose state. For loose sand the angle of friction does not vary much with the pressure, and if the Author's suggestion is correct, no reduction in the rate of increase of pressure with breadth would be expected for tests on loose sand.

## Comparison between Calculated and Measured Bearing Capacity on Sand.

The modified Prandtl formula [4, 5] for the bearing pressure $q$ of a footing of width $2b$ is :—

$$q = \left[c \cot\phi + \gamma b \tan\left(45^\circ + \frac{\phi}{2}\right)\right]\left[\tan^2\left(45^\circ + \frac{\phi}{2}\right)e^{\pi \tan\phi} - 1\right]$$

where $\gamma$ denotes the weight of the foundation material per unit volume, $c$

its cohesion, and $\phi$ its angle of internal friction. On a purely frictional material, such as clean, dry sand, $c$ is equal to zero.

In order to determine the value of $\phi$ for the sand, shearing tests were carried out in a shearing box apparatus, under various normal pressures. In order to prevent jamming of the box and piston during the tests, it was found necessary to remove the coarsest and the very fine particles from the sand. This was done by sieving the sand through 2-millimetre and 0·15-millimetre sieves and using the fraction passed by the former and retained on the latter. The grading curve for the sieved sand is shown dotted in *Fig. 3*. It is not considered that the small difference between the two grading curves will noticeably affect the result of the shear tests.

The sand was vibrated in the shear box in order to obtain a packing of the grains as dense as that used in the loading tests, but this was not found possible, and the density of packing obtained corresponded to a voids-ratio of 0·56. This fact is explained by the greater restraints which existed in the shear box in comparison with the large wooden box, in which the sand was able to move more freely and so adopt a closer packing. The shear-strength curve in the loading tests might, therefore, be expected to be slightly higher than that for the shear tests.

The results of the shear tests are given in *Figs. 8* and *9*, together with tests on the same sand in a loose condition. It will be noted that the angle of friction $\phi$ is not a constant for the material, but varies with the normal pressure $p$ and with the initial voids-ratio $\epsilon_i$.

This variation of the value of $\phi$ with normal pressure renders the determination of the value to be used in the Prandtl formula a matter of some difficulty. In the loading tests the actual bearing pressure is known, and therefore the average normal pressure on a typical surface of failure can be estimated by means of the theory of elasticity. The order of this pressure is approximately half the bearing pressure, that is, about 1·3 ton per square foot at failure. The value of $\phi$ corresponding to this pressure is shown by *Fig. 9* to be 44·4 degrees, which value, when substituted in the modified Prandtl formula, gives an ultimate bearing pressure of 3·5 tons per square foot. This value is considerably higher than the observed value of 2·6 tons per square foot. It is instructive, however, to plot out the variation of $q$ with $\phi$ according to the Prandtl formula, as has been done in *Fig. 10*. It will be seen that when $\phi$ is greater than 40 degrees the curve is very steep, and that only a small error in $\phi$ is required to cause a large variation in $q$. The value of $\phi$ corresponding to the observed bearing pressure is seen from *Fig. 10* to be 42·6 degrees, which is a quite possible value for $\phi$.

The normal use of the formula, however, is to calculate the ultimate bearing pressure, and in this case the pressure on the surface of failure will be unknown. However, for most natural soils the values of $\phi$ will not be so high as those for the clean compact sand used in these tests, nor will the density be so great. For these reasons the variation of $\phi$ with

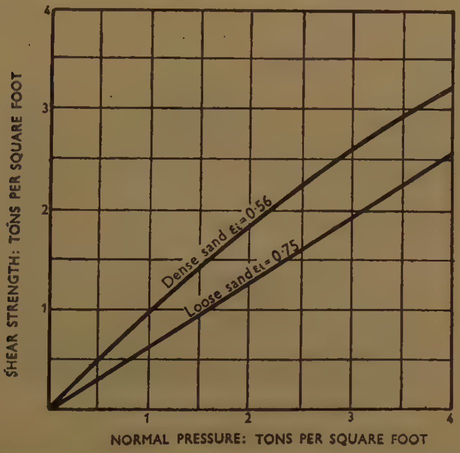
normal pressure is likely to be less, and the value of $\phi$ chosen for substitution in the formula will not be so critical. In most cases, therefore, it should be possible to choose a lower limit for $\phi$, which, when substituted in the formula, will furnish a reliable lower limit to the ultimate bearing pressure.

Although the agreement between the observed value of the ultimate bearing pressure and that calculated from Prandtl's formula is not exact, both values are of the same order. This is not the case when Ritter's [6] formula is used. Ritter's formula for the ultimate bearing pressure of a footing of width $2b$ on the ground-surface is :—

$$q = \left[c \cot \phi + \tfrac{1}{2}\gamma b \tan\left(45^\circ + \frac{\phi}{2}\right)\right]\left[\tan^4\left(45^\circ + \frac{\phi}{2}\right) - 1\right]$$

The curve of $q$ against $\phi$ is plotted in *Fig. 10*, showing that the calculated

*Fig. 8.*




VARIATION OF SHEAR STRENGTH WITH NORMAL LOAD.

bearing pressure for the probable range of $\phi$ is from 0·4 to 0·5 ton per square foot, which value may be compared with the observed value of 2·6 tons per square foot. The value of $\phi$ required to give the observed bearing pressure is 56·7 degrees, which is an impossibly high figure.

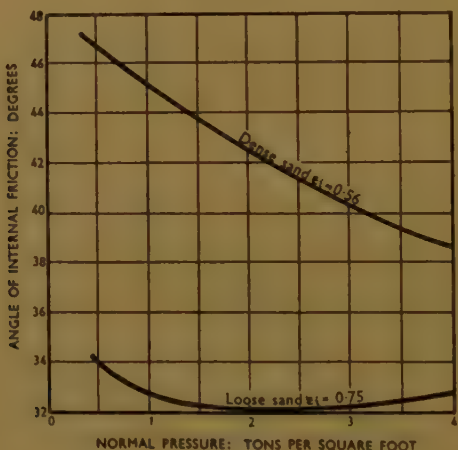
In Krey's graphical method of analysis [7, 8], the surface of sliding is assumed to be a circular arc under the footing, terminating in a tangent at an angle of $\left(45 - \frac{\phi}{2}\right)$ degrees to the ground-surface. The centre of the most dangerous circle is found by trial. Krey states that this centre lies on the level of the under side of the footing, and he takes trial centres only on this level, a method indicated also by Plummer[8]. In the calculation of the ultimate bearing pressure for the loading tests the centre was

also varied vertically and the worst centre was found to be above the level
of the bottom of the footing.

Krey's method gives an answer which agrees with the observed value
when $\phi$ is assumed equal to 44 degrees. This value is practically identical
with that obtained above by estimating the mean normal pressure on the
surface of failure. Krey's method would therefore appear to be the most
reliable. The close agreement between the bearing pressure obtained by
the Krey and the Prandtl methods for various values of $\phi$ is shown in
*Fig. 10.*

On one point a radical disagreement appears between the calculations
and the experiments. This is in the position at which the surface of
shear failure cuts the top surface of the sand. The observed position

*Fig. 9.*




VARIATION OF ANGLE OF INTERNAL FRICTION WITH PRESSURE.

varies between 10 inches and 18 inches from the edge of the footing, wherea
the theoretical position in both Prandtl's and Krey's methods is wel
outside the box used in the tests. This difference does not seem to b
due to any restraining effect of the box, since it occurs to the same exten
with the 3-inch footings, but is due rather to a difference between the actua
mechanism of failure and that assumed in the analysis. A further stud
of the mechanism of failure is proceeding at the Building Research Statior

It is interesting to note that the value of $\phi$ in Meischeider's test result
calculated according to Prandtl's equation, ranges from 35 degrees t
40 degrees. This appears to be a more likely range for the dense san
used by Meischeider than the value of $32\frac{1}{2}$ degrees which he uses in h
calculations. Meischeider develops a three-dimensional method o
calculation of the ultimate bearing pressure, which, he asserts, exhibi

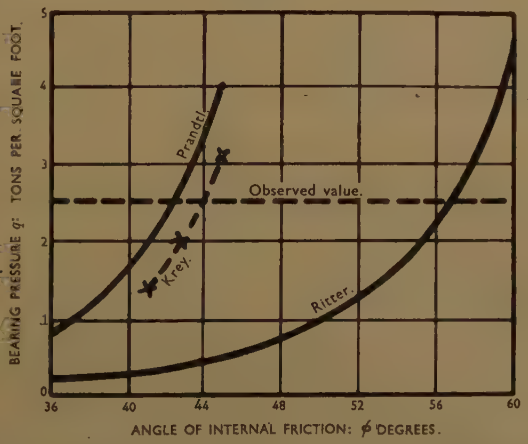
good agreement with his test results. However, this method gives an ultimate pressure for a square which is about 30 per cent. lower than for a rectangle of equal width—a result which is not borne out by the tests described above.

## Bearing Tests on Clay.

No determinations of ultimate bearing pressure on various sizes of footings on clay appear to have been published.

As stated above, theoretically, breadth has no influence upon the bearing pressure of a footing on a plastic material such as soft clay, the bearing pressure depending only upon the shear strength of the material

*Fig. 10.*




Variation of Bearing Pressure with Angle of Friction.

The theoretical bearing pressure, however, is calculated for an infinitely long strip footing, and it is desirable to know the relationship between such a load and a square footing of equal width. A few tests were carried out on two footings, 18 inches by 3 inches, and 3 inches by 3 inches, on clay, in order to obtain some indication of this relationship. The clay used was London Clay which had been thoroughly re-moulded with water and stored in a bin for several months. The clay was placed in a wooden box 2 feet 6 inches by 1 foot 6 inches and 8 inches deep, in layers, and thoroughly rammed with wooden rammers 2 inches square. The top surface was then carefully levelled off. The footing was placed in the centre, and was forced into the clay at a constant rate by a screw-driven testing-machine. Load and settlement readings were taken during test, and after each test samples of the clay were taken for shear test and moisture-content de-

terminations; the clay was then thoroughly re-moulded, and again rammed into the box and levelled off for the next test.

In all, three tests were carried out on each footing. The variations between the three tests are shown in Table II.

TABLE II.

| Test No. | 1. | 2. | 3. | 4. | 5. | 6. |
|---|---|---|---|---|---|---|
| Size of footing: inches . | 3×3 | 18×3 | 3×3 | 18×3 | 18×3 | 3×3 |
| Ultimate pressure: lb. per square inch . . . . | 12·76 | 9·86 | 12·50 | 9·64 | 9·71 | 12·30 |

The mean values of the ultimate pressure were 12·5 lb. per square inch for the 3-inch by 3-inch footing, and 9·7 lb. per square inch for the 18-inch by 3-inch footing. Therefore it would appear that for footings of this size on a plastic foundation material, the ultimate pressure on a square is approximately 30 per cent. higher than on a long rectangle of equal width.

## Comparison of the Calculated and the Observed Bearing Pressure on Clay.

A plastic material is one with no internal friction and with a constant shear strength $c$ due to cohesion. For such a material the formulas of Prandtl and Ritter reduce to very simple expressions, namely, $q = (2 + \pi)c$ and $q = 4c$ respectively. A soft clay approximates to a plastic material when tests are carried out on it under such conditions that there is no increase of shear strength due to consolidation of the clay. In such a case it is better to write $s$ (existing shear strength), rather than $c$ (cohesion), in those formulas.

From seven shear tests carried out on the clay, the mean shear strength was found to be 1·88 lb. per square inch, the mean moisture-content for these tests being 42·2 per cent.; the mean of thirteen independent moisture-content determinations was 42·4 per cent.; and the liquid limit for the clay was 74·2 per cent.

If the mean bearing pressure for the long rectangle is expressed in terms of the shear strength of the clay, a value of 5·1 times the shear strength is obtained, which is almost identical with Prandtl's value of $(2 + \pi)s = 5{\cdot}14s$. For the case of a plastic material, Wilson [9] has obtained a mathematical solution for Krey's graphical method. According to this, $q = 5{\cdot}41s$, which also agrees closely with the observed value. Ritter's formula, $q = 4s$, gives too low a result.

The case of a square footing has not been solved theoretically; but Hencky [10] gives the ultimate bearing pressure of a circle on a plastic

foundation as 5·6$c$, which is higher than Prandtl's value for the strip. From the tests the ultimate pressure on a square is 6·7 times the shear strength, which also shows an increase on the strip.

The limitations of these few tests are realized: but further evidence which points in the same direction was recently obtained by the Building Research Station from the analysis of the failure of a footing 8 feet by 9 feet on soft clay[11]. The bearing pressure calculated from the formulas given above agreed reasonably with the load which caused failure.

## Conclusions.

A study of the test results of the various investigators, together with those described in this Paper, permits the following conclusions to be drawn:—

For footings on sand the ultimate bearing pressure is approximately proportional to the width of the footing, but when the sand is dense the rate of increase of pressure decreases as the width of the footing increases.

The maximum pressure on a square footing on sand is equal to that on a long strip of the same width.

Prandtl's formula and Krey's graphical method of analysis give results which agree closely with the observed bearing pressure for small footings. Although, in practical cases, difficulty may be experienced in choosing the correct value of $\phi$ to be used in the formula, a lower limit can always be obtained. In many cases, the high values of bearing pressure calculated for large footings on frictional soil, will be ruled out by considerations of settlement.

The mechanism of failure on sands is not completely understood, and further work on this point is proceeding.

For small footings on plastic clay the ultimate bearing pressure of a square is approximately 30 per cent. higher than for a long strip of equal width: again, the values of bearing pressure given by Prandtl's and Krey's methods agree well with the experimental results for a long strip.

## Acknowledgements.

The Author is indebted to his colleagues at the Building Research Station, and to the engineering staff at the Imperial College of Science and Technology (where the loading tests on sand were carried out) for many helpful suggestions and generous assistance.

The Paper is published by permission of the Director of Building Research.

The Paper is accompanied by two sheets of drawings and three photographs, from which the Figures and the half-tone page plates in the text have been prepared.

## APPENDIX.

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Paper No. 5289.

# "Psychology of Steering in Relation to Road Curve Design."

By Professor FREDERICK GEORGE ROYAL-DAWSON, M. Inst. C.E.

*(Ordered by the Council to be published in abstract form.)*[1]

IN view of the prevailing uncertainty as to the particular unit which should be adopted as the standard rate of change of centripetal acceleration, usually denoted by the symbol $c$ in the design of road transitions, and as to the meaning of which much misconception exists, the purpose of this Paper is to clarify the position by recourse to first principles, both psychological and dynamic, upon which the whole subject depends.

Acquaintance with the definitions of $c$ and of "unit-chord", as used in the Author's treatise on "Curve Design",[2] is assumed, but a memorandum is included in the Paper for convenience of reference.

It is shown that the keynote of experienced driving is economy of effort, and that there is considerable differentiation between light and heavy vehicles from the driver's point of view. These are classed as "Type A" and "Type B" vehicles respectively, and for facilities of comparison are assigned definite dimensions, Type A being represented by a 1-ton car, having a wheel-base of 8 feet, a locking angle of 30 degrees, and a steering-gear ratio of 6 : 1, whilst Type B, weighing about 10 tons, is given a wheel-base of 16 feet and a steering-gear ratio of 12 : 1, with a locking-angle of 30 degrees, as for Type A.

The mechanical limitations of both types in confined spaces are illustrated, and the relationship between "fugitive" and "pursuit" curves, as exemplified by the steering and rear wheels respectively, is demonstrated at length by characteristic wheel-tracks.

The personal factor is discussed, and it is shown that the operation of steering is usually effected by a series of tentative hand movements or "strokes", short or otherwise, linked by pauses or "rests" of appropriate duration, according to the nature of the curve to be followed, each stroke connoting a fragmentary transition and each rest a circular arc. It is shown graphically that a given transition may, geometrically speaking, be very approximately followed by a motorist with a fairly wide range of

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[1] The full MS. and illustrations may be seen in the Institution Library.—SEC. INST. C.E.

[2] "Elements of Curve Design for Road, Railway, and Racing Track." Spon, London, 1932.

spontaneous hand movements on the steering-wheel, and that it is then only a matter of routine steering to ensure that the resultant curve conforms reasonably to the general course of the road.

The visual aspect of steering is considered, and the Author shows that the "fugitive" and "pursuit" principles which describe the inter-relation between the steering and rear wheels are applicable in modified form to the general operations of steering in relation to road objectives. After enumerating these, the principles are illustrated by specific examples of moderately sharp road curves, in two series, (1) wholly circular at bends of 90 degrees and 45 degrees respectively, and (2) wholly transitional, at corresponding bends.

A very complete analysis is made of all possible vehicle-tracks within these lay-outs, from which the conclusion is drawn that transitional lay-outs impose definitely less physical effort on the driver than does the circular type, so far as sharp curves and moderate speeds are concerned, and that the same principles would apply, although less obtrusively, with increase of radii and speeds.

The dynamic aspect is dealt with, and the Author shows that the inherent physical effort in steering a heavy "Type B" vehicle, combined with the necessity for turning the steering-wheel through an angle four times as large as that required by an "A" type car over the same curve, renders it imperative to design curves to meet the exacting requirements of heavy vehicles instead of adopting standards fit only for light cars, as is too frequently the practice.

The Author demonstrates that, whilst the driver of a heavy vehicle can, on occasion, steer his vehicle over a given curve at a speed equivalent to a $c_2$, and even higher, standard, he will not do so if he can possibly find an alternative course involving a lower value of $c$, whether by a "short-cut" on to a neighbouring lane, or by other means. Drivers of light cars are prone to adopt similar expedients, although higher values present no difficulties to them. Examples are given in support of these contentions.

Personal reactions to various values of $c$ are discussed, particular attention being given to the sensations produced by a motor-coach making a right-hand turn over a badly designed roundabout, at speeds of 10, 12·5, and 15 miles per hour respectively. Among other examples cited is the case of a motorist making an unsuccessful attempt to effect a sharp left-hand turn from a main highway into a side road at too high a speed of approach, resulting in a collision with the corner of a wall. This is attributed primarily to the inconspicuous entrance to the side road, obstructed by an unduly projecting pavement on the near side, thus illustrating a common error in road design.

The foregoing investigations are confined to moderately sharp curves and comparatively low speeds. For large radii and high speeds, a different set of psychological and dynamic factors are brought into play. It is inferred that the higher the required standard of speed, the lower should

be the standard value of $c$ for design purposes, in order to provide the requisite margin of safety.

The Author presents general conclusions, of which the principal are the following:

Although both light and heavy vehicles can, on occasion, attain $c_3$, and even higher, standards imposed on them by bad design or sudden emergencies, such standards should be ruled out as a basis of design in the interests of public convenience, and the design should be restricted to a choice between $c_1$ and $c_2$.

As the distance from the apex to the mid-point of a curve is frequently determined by topographical considerations, limitations are thus imposed on the radius at the mid-point, which, in turn, imposes limitations on the speed-value with reference to a given maximum centrifugal ratio, such as 0·25, by whatever standard the approach transition is laid out.

In general, for a given value of $c$, a wholly transitional curve, when such is practicable, has a greater speed-potentiality than any possible combination of transition and circular arc passing through the same mid-point. It is also evident that the wholly transitional curve gives a mathematically shorter route than any such combination, measured from some point on the tangent to the given mid-point. From the point of view of the commercial user, therefore, it is also the most economical to work.

It is shown that advocacy of the $c_2$ in preference to the $c_1$ standard emanated from the United States, where it has perhaps been artificially fostered by association with the "spiral-degree" method of setting out transitions, which is the only recognized method of laying out such curves in that country. This method is inherently unsuited for laying out wholly transitional curves, as it involves a central circular arc of convenient "degree" value in all curves without exception. Thus, it may be conjectured that the desire for short transitions (suggesting the $c_2$ standard) was primarily based upon their supposed value in contributing to simplicity of construction. As the Author's "unit-chord" system is free from the handicap thus imposed by the "spiral-degree" method, the advocacy of the $c_2$ standard as a means of securing short transitions is deprived of its force as an open argument either for the simplification of construction or for the supposed desirability of short transitions, as such, on their own merits. It is, however, shown that the majority of transitions designed for $c_1$ speeds will take $c_2$ speeds (that is $+$ 0·26 or about one-quarter more) should occasion arise, without violating centrifugal considerations, and that $c_2$ speeds on a given transition may often attain $c_1$ attributes by adopting "short-cuts", encroaching on neighbouring lanes, in the absence of adverse traffic. The Author therefore suggests that, as a general practice, in recording the speed-value of a given curve, the basic value should be calculated on the $c_1$ standard, as being the more comprehensive, with the addition of approximately one-fourth in brackets, to

represent the equivalent $c_2$ value on the same curve, thus, 35(44). This would convey the meaning that on roads of light traffic designed on $c_1$ principles, $c_2$ speeds (in brackets) would generally apply, but that with increase of traffic the speed-value would be automatically reduced to the basic $c_1$ standard. He points out, in this connexion, that in the United States, where the $c_2$ standard prevails, there are fewer vehicles per mile, and also a smaller proportion of heavy-type vehicles, than in Great Britain.

The Paper is accompanied by five sheets of drawings.

## "INGENUITY" COMPETITION, 1941.

# "The Sanitation of Tube Railway Stations used for Air-Raid Shelter."

By LEONARD BUSHBY ESCRITT, Assoc. M. Inst. C.E.

WHEN one decides to employ a primitive method in engineering, one stands the chance of gaining the interesting experience of exploring not untrodden, but forgotten, ground. The following Paper describes the investigation of a method of sewage ejection which led to the rediscovery of the difficulties that must have been surmounted by the first inventors of the sewage-ejector: but the methods adopted to overcome such difficulties differ from those utilized in commercial designs.

## THE PROBLEM.

When the London tube railway-stations were opened to the public as dormitory air-raid shelters, the provision of sanitary conveniences became essential. The London Passenger Transport Board provided numbers of chemical or pail closets and arranged temporary closet screens. The local authorities undertook the emptying and cleansing of the closets. The latter was not a simple operation. The difficulty of manhandling the closets, or the larger receptacles into which the closets were tipped, was augmented by the fact that in most cases the closets or receptacles had to be carried up flights of stairs, along access passages, and to lifts or escalators, before being tipped into the nearest water-closet or manhole. The work involved considerable expenditure on labour, and it was not long before the advice of the Ministry of Home Security was sought.

On the 28th September 1940 the Author was instructed to find a solution to the problem of the removal of chemical closet sewage from platform-level to the nearest sewer on all tube stations which were occupied by the public sufficiently to warrant moderate expenditure. On the 30th September he proposed a scheme which was approved, and on the same day financial sanction was obtained for the work to be put in hand. By the 2nd October the engineers of the London Passenger Transport Board had agreed to provide the assistance of their architects' department for the preparation of surveys and plans and the letting of contracts, and within a short period six contracts were let for installation and four for the manufacture of plant.

## Data and Local Conditions.

All except a very small minority of London tube stations suitable for shelter purposes are at depths below ground-surface which make possible the removal of sewage either by single-stage unchokeable pump or by air-ejector. Apart from pumping methods, either the lifting of tanks up vertical shafts or the handling of special containers up escalators could be considered. The former alternative, however, is ruled out by the rarity of vertical shafts discharging into convenient positions at the surface, and the latter method by the complexity of the routes from the stations to the surface and the amount of labour involved. One is therefore left with the methods of ejection and pumping.

Air at a maximum pressure of 60 lb. per square inch, and a theoretical minimum of 45 lb. per square inch, is provided at all stations as part of the normal equipment of the tube railway system. As ejectors, when installed without compressors, cost less than pumps, there was good reason for their selection in this instance. When, however, the Transport Board engineers were approached on this subject, they agreed to supply air, but restricted the supply to the discharge of a ⅛-inch diameter orifice at each station. It was calculated that an orifice of this size would, unless an air-bottle were provided, give a rate of discharge from an ejector that would produce a far from self-cleansing velocity in any rising main of diameter sufficiently large to avoid choking (that is, 3 inches or 4 inches internal diameter), and that therefore there would be danger of silting, not only in the rising main but also in the ejector. Silting of the rising main was a matter that had seriously to be considered, as in many stations a continually rising gradient could not be maintained from the ejector to the surface, and the rising mains would be long, with many bends. Moreover, the sewage to be ejected contained more solid matter than a normal sewage, being comparable, in solid content, with sewage sludge. From an estimate made by a local authority at a largely-occupied station, the average quantity of sewage was 2·9 pints per head per night of 12 hours approximately. This can be compared with a commonly-experienced proportion of sewage sludge, namely, ½ gallon per head per day of 24 hours, with a solid-content of 2½ per cent. It had been reported that all kinds of waste, including tins, cigarette-packets, broken bottles, and cast-off clothing, in other words the worst material to pass through an ejector or pump, had been found in the closets.

An important factor, and the main one which decided the choice of installation, was the necessity for quick completion of the work in order to avoid further waste of money on labour. This meant that delay due to waiting for the manufacture of machinery had to be avoided as far as possible. Eighty stations and one hundred ejector plants had to be considered. Not many firms manufacture sewage-ejectors, and therefore quick deliveries of machinery could not be expected. As the plant would

have only a temporary value, and would have to be dismantled at the end of the war, low cost would have to take precedence over refinement.

## The Method Adopted.

As no existing manufactured article fitted all the requirements, it was decided to improvise. Crude hand-operated ejectors were to be used, with bodies or tanks made of mild-steel sheet by tank-manufacturers, and, as far as possible, second-hand containers or boilers were to be employed. (These would cost no more than the air-bottles necessary for automatic ejectors.) Large capacity of the ejectors would reduce operation labour, and therefore a size that would in most cases require not more than two ejections per night was chosen.

No time was available for detailed preparation of plans, but a rough pencil sketch was made and prints were circulated to manufacturers, with the briefest of specifications. Apart from this there were no working drawings other than small-scale layout plans drawn by the architects' department.

## Principles of Operation.

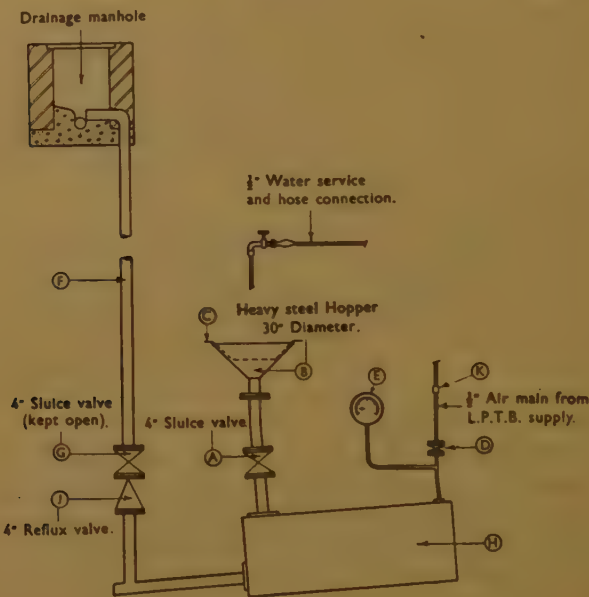
*Fig. 1* illustrates a typical plant in its simplest from. The contents of chemical closets are tipped through a metal screen C, which retains large objects liable to injure the plant, into a steel hopper B, whence the sewage gravitates to a 200-gallon tank H. When this tank is nearly, but not quite full, an operative closes the sluice valve A and opens the air-valve D. The air then flows through the ⅛-inch nozzle K into the tank and propels the sewage slowly up the rising main F until the tank is nearly empty. When the tank is sufficiently empty for air to enter the rising main the flow increases, and the plant then combines the qualities of an ejector and an air-lift, for the air in the rising main lightens the column of sewage, reducing the manometric head, with the result that the compressed air remaining in the tank rapidly blows out the last of the sewage, together with all solids, clearing both tank and rising main of sediment. In *Fig. 1* the rising main is shown coming off the tank near the bottom. Actually, the arrangement was that the rising main should come off right at the bottom, so that the tank, which was inclined towards the outlet, should be completely emptied. The sluice-valve G and the reflex-valve J were inserted to provide the usual precautionary measures against back-flow after discharge, and to prevent blow-back of foul air from the sewers.

It was at first intended that the operative should be warned when the tank was empty by the noise of the rush caused by the sudden release of air at the end of ejection. In practice, however, it was found that the tank could seldom be placed in a position for this to be possible, and the pressure-gauge shown in *Fig. 1* was added, the principle being that the

gauge would show the fall of manometric head on the release of air up the rising main, indicating the completion of discharge.

A further modification, not shown in *Fig. 1*, was the addition of an air-release valve mechanically coupled to the valve D in order that on

*Fig. 1.*




turning off the air-supply any residual pressure in the tank would be released, thus preventing any risk of blow-back through the hopper.

## Behaviour in Practice.

In testing the installations the time of discharge was recorded, together with the maximum steady reading on the pressure-gauge. These records were taken purely for the purpose of ascertaining that the discharge took place in a reasonably short time and that the pressure-gauge functioned satisfactorily. As the reading on the pressure-gauge indicated the manometric head, the time of discharge was an indication of the difference between the pressure in the air mains and the pressure in the ejector. Generally, it would be expected that short times of discharge would accompany low readings on the pressure-gauge. If the pressure had been constantly at a maximum of 60 lb. per square inch in the mains, this would have been so, and a curve of pressure against time of discharge could have been plotted. Actually, considerable local variations of air-suppl

pressure occurred, the values of which were not known, with the result that the pressure-gauge readings were chaotic. The highest pressure recorded in a test accepted without reservation was 45 lb. per square inch and the lowest 16 lb. per square inch. At least 75 per cent. of the ejectors completed their discharge in from 5 to 10 minutes.

In the first installation to be completed and tested the lift was moderate, whilst the rising main sloped upwards at an angle underneath an escalator and discharged into a medium-sized manhole. The test was satisfactory, discharge taking place in a sufficiently short time. The only fault found was that, owing to the drain into which the rising main discharged being silted, there was moderate flooding at a downstream gully. This was quickly remedied. In similar installations, and in those in which the rising main was carried horizontally beneath the platforms for some distance, the results were, in the main, satisfactory; but when a further plant was tested, in which the rising main, erected in the shaft of a spiral staircase, was vertical, it was found that the final rush due to release of air was very violent and caused the blowing of water-seals in gullies and closets near to the manhole into which the rising main discharged. In most cases the trouble could be overcome by simple expedients, for example, directing the end of the rising main so as to produce a jet playing into the outlet of the manhole, or else merely adding an air-vent, but about 10 per cent. of the cases proved more difficult of treatment.

To appreciate the difficulty, one must understand exactly what happened at the end of the ejection. Taking as an example a case where the manometric head was 45 lb. and the time required for pressure to fall from 45 lb. to zero was $\frac{3}{4}$ minute; in that time roughly 30 cubic feet of compressed air, or 90 cubic feet of free air, would be expelled from the tank. Assuming that the 4-inch rising main was 120 feet long, the released air would mingle with the 10 cubic feet of water retained in the rising main, blowing it out as foam. The resulting discharge at the upper end of the rising main would be 133 cubic feet per minute of spray, at a velocity of about 27 feet per second. Thus, it is not surprising that it was found that, before remedies were applied, rising mains not sufficiently stayed were visibly deflected by the violence of the discharge, traps were blown, and even manhole covers lifted.

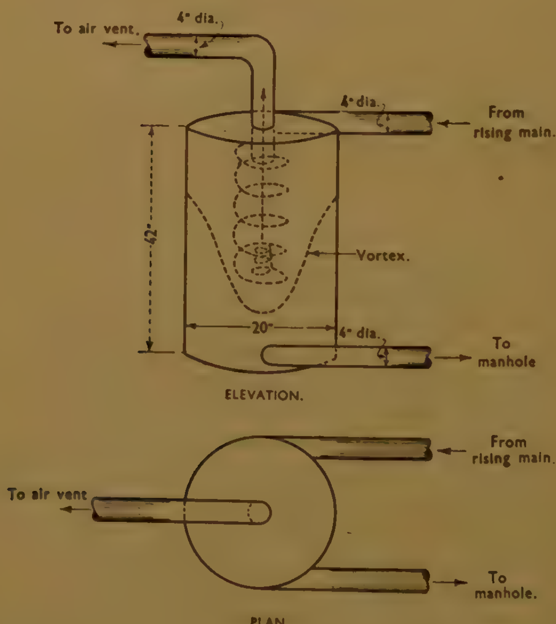
## Separator Tank.

A well-vented large manhole would often absorb the rush of water and air; but in the more modern tube stations cast-iron pipes are laid with hatch-boxes, or the manholes are small and shallow. In the first experiment, to take the place of a large manhole a separator tank, consisting of a rectangular hot-water tank of about 50 gallons capacity, fitted with inlet, outlet, and air vent, was placed so as to receive the discharge of the rising main, gravitate the sewage to the drainage system, and release the air to a

ventilating pipe. Sometimes this was satisfactory, but not always, for in some cases the contents of the rising main were too great and too quickly discharged to be accommodated by a small tank, and spray was blown out of the vent-pipe. To remedy these defects larger tanks were used, but none larger than 100 gallons was found to be necessary.

The most satisfactory result was obtained with the aid of a circular tank in which centrifugal action was employed in an arrangement involving the principle of the cyclone dust-collector, and this form was adopted for all further cases. The tank was constructed as illustrated in *Fig. 2.* The

*Fig. 2.*




rising main was brought into the top tangentially and the outlet was arranged tangentially at the bottom. Thus the sewage was caused to rotate violently in the tank, forming a vortex. The air was released from a pipe projecting through the top of the tank centrally over the vortex and carried down to the level of the invert of the incoming rising main. Following the principle of the cyclone dust-collector, the air released from the water in the hollow of the vortex would rotate rapidly and centrifuge its spray outwards. The air would then pass free of water-particles to the vent-pipe and the water would gravitate free of air to the drainage manhole. This type of tank was found to be unsatisfactory only when not of

sufficient size to prevent it from becoming surcharged. In all cases the tank was set in concrete to prevent movement.

## Effects of Air Lock: General Observations.

In a few instances minor air-locks formed in the rising mains, but none necessitated air-release valves. It was found that they did not interfere noticeably with discharge, although it appeared that separation of water and air occurred in the U-tube during the final ejection, causing a series of two or three discharges of air-free water alternating with water-free air.

A few practical observations were made which would affect the design of any further plant on the same lines. Reflex valves were found to be unreliable except for preventing the flow of foul air. The smallest chip or shaving was sufficient to permit audible back-flow of sewage; for this reason the rising main should never have a capacity greater than that of the tank.

Sluice valves were found reliable for retaining air. It was found that by far the most satisfactory arrangement was to insert them in the rising main or below the hopper in such a position that they could close horizontally. In the vertical position it was possible for large metallic objects or broken glass dropped into the hopper to lodge in the recess below the gate, preventing closure.

Although 4-inch rising mains were used in nearly all instances, 3-inch mains were found to be quite unchokeable, and were less liable to permit violent final ejection.

What was perhaps the most interesting possibility was not tested, namely, the intentional blowing of air into the rising main at a point some distance above the ejector, so as to lighten the column of sewage, and render possible ejection where the total lift would normally be too great for the air-pressure available. It was this combination of air-lift and ejector principles that gave the problem its novelty and interest, and for that reason it is particularly regretted that the circumstances of the work did not permit leisurely study, exact scientific investigation, or the use of flow-recorders, or other instruments, whereby an empiric theory might have been devised.

The Paper is accompanied by two sheets of diagrams and two photographs, from which the Figures in the text have been prepared.

## INGENUITY COMPETITION, 1941.

# "Repair Work to a Damaged Pumping-Station."

By Stephen Verrall Gardner, Assoc. M. Inst. C.E.

### General.

As a result of enemy action the main pump-house of a power-station was put out of action. Had it not been for the existence of an emergency pump in an adjacent building, the supply of cooling water to the condensers of the power-house turbines would have ceased, resulting in a temporary break in the whole output of electrical power from the station.

### Damage.

The damage to the pumping equipment and to the pump-house was more serious than appeared at first sight.

Structural damage to the pump-house consisted mainly of horizontal fractures to the columns, both above and below the motor floor level, whilst one wall panel below ground-level was completely removed. A small area of the basement floor was damaged, and one pump foundation was slightly tilted. The three vertical pumping units were thrown out of alignment and one of the pump casings was fractured.

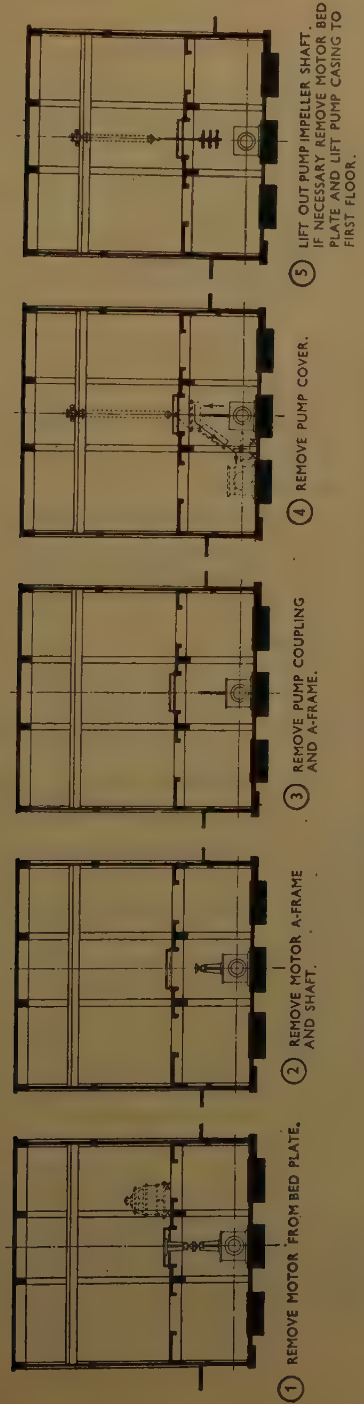
More serious developments followed when the building began to settle at one corner. Although the settlement was slight, the question of re-alignment of the machinery on the existing structure was rendered rather problematic, as it could not be guaranteed that further settlement was out of the question.

As the building was founded on heavy yellow clay, below water-level, it would not have been easy to deal with this question of settlement within a short time.

The most urgent problem to be solved, therefore, was that of getting the motors and pumps re-aligned and working again in the shortest possible time, without the danger of further trouble developing from the same source.

As the motors were bedded on the ground floor, whilst the pumps were founded on the basement floor, the problem resolved itself into the development of some scheme whereby the motors and pumps would remain correctly in line and in running order, even though the building should subside, or the upper floor move relative to the lower floor. This in itself

*Figs. 1.*




SEQUENCE OF OPERATIONS FOR DISMANTLING MOTOR AND PUMP UNITS.

presented many difficulties, as the building was nearly 20 years old and was completely of reinforced-concrete construction. Moreover, very little space was available in the basement between the pumps and the supply main in which to place new structural members. Further, it was necessary to dismantle the motors and pumps annually, and often more frequently, to carry out the usual maintenance overhaul.

This process was sufficiently complicated in the limited space of the existing pump-house, as is evident from *Figs. 1*. Thus the development of a satisfactory solution entailed considerable thought.

## CONDITIONS.

The final solution would also have to satisfy the following conditions, some of which include restrictions previously mentioned :—

(*a*) Rigid vertical and lateral connexion between the pumps and motors had to be provided.

(*b*) The minimum of space was to be occupied by any new structural members, allowing also for space required for the dismantling process.

(*c*) The pump-house had been scheduled for demolition as a result of the damage ; hence it was unwise to spend too much time and money on the building, and attention had to be concentrated rather on getting the machinery re-aligned on rigid foundations so that the one pump then doing full duty could be relieved.

(*d*) Structural steelwork was undesirable owing to the damp atmosphere.

(*e*) The repair work would have to be done in stages, and one pump temporarily reset so that it could be called into service in an emergency.

(*f*) The minimum of excavation around the existing foundations, or of breaking away of the motor-floor, was advisable, owing to the age of the building, its damaged condition, and other factors.

(*g*) The size of the original floor openings could not be reduced by more than a few inches, as it was still desirable that the pump casings could be lifted on to the motor-floor if necessary.

(*h*) During subsequent overhauls, one or two of the three motors would be removed, either separately or at the same time, and this would produce loading of a non-symmetrical character on the new work. This had to be provided against, so that the remaining unit or units could still function efficiently.

Several proposals were made to meet this problem, but after re-shaping to suit the governing conditions, only one method appeared to be able to overcome the restrictions efficiently and economically; therefore it was decided to isolate the three vertical pumping units, and to provide separate new supports, so that these could be independent of any settlement in the main building.

This could be done by the construction of a reinforced-concrete cradle founded on the existing pump bases, and carrying the motors over, independent of the motor-floor.

Certain portions of the motor-floor, however, would have to be broken away if this scheme were adopted, whilst some excavation would be necessary in order to bond the new column foundations to the existing pump bases. A compromise could be made with respect to the latter, resulting in the new column foundations resting partly on the existing pump bases, whilst a small portion of floor-slab could be removed between them, so that the new work would have adequate lateral restraint.

It appeared inevitable that some parts of the motor-floor would have to be broken away to accommodate the new work, but the disposition of the existing main floor beams was such as to render this a comparatively simple matter.

A trial hole taken out between one pair of pump bases to investigate the depths to which these were taken, revealed that a certain quantity of water would have to be contended with during the actual construction of the new foundations, although these penetrated the existing floor by only about 9 inches.

## Solution.

The final solution, illustrated by *Figs. 2* to *5*, consisted in erecting eight reinforced-concrete columns to carry the motor cradle. These columns were founded on four separate concrete foundations, which were carried partly on the existing pump bases and partly on the ground between them. They were kept well out of the way of the pumps, leaving sufficient room for the latter to be dismantled. In spite of this, eight of the twelve corners of the three motor-beds were provided with direct column support.

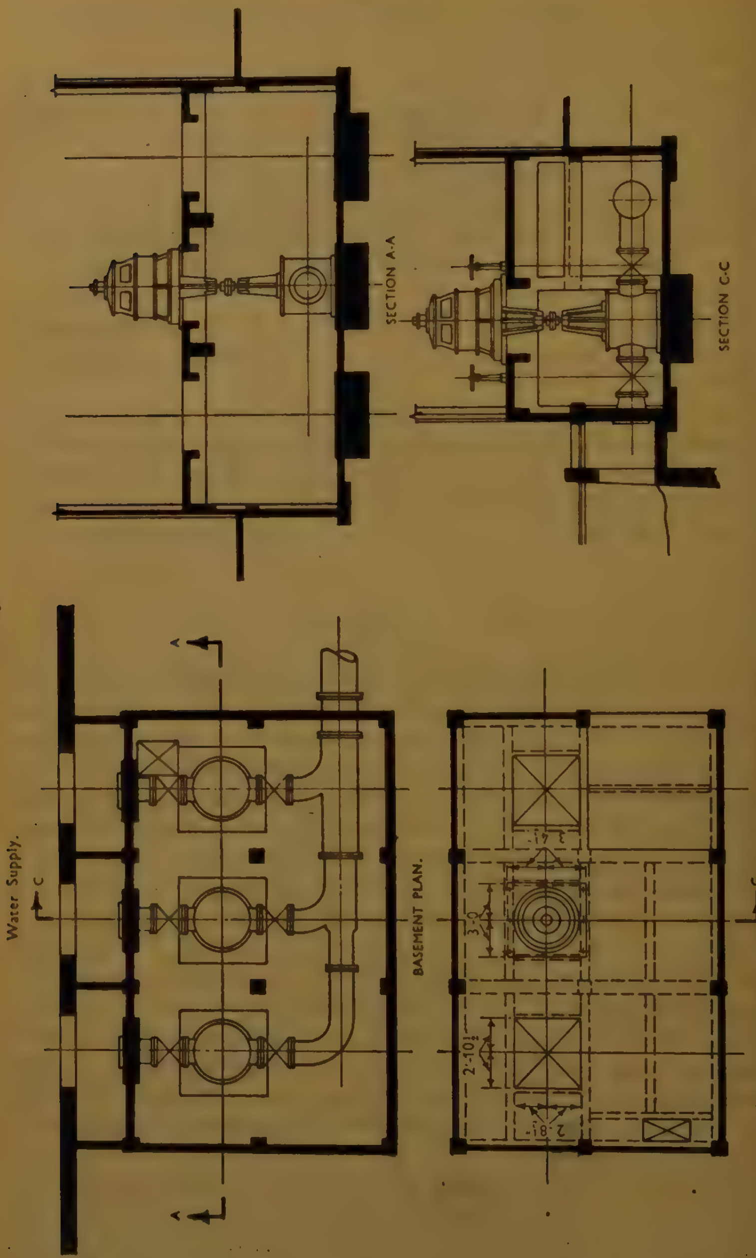
The remaining two pairs were supported by concrete brackets carried by reinforced-concrete beams spanning between the middle two pairs of columns.

Lateral connexion between the columns was obtained by the introduction of two tie-beams, above the motor-floor level, one on each side of the motor bed plates, which were turned round from their original position.

Seatings were formed for the motor bed-plates by means of angles around the four edges of each opening, and these were carefully levelled up when being fixed so that the final setting of the motor bed-plate on the angles would not be a tedious job.

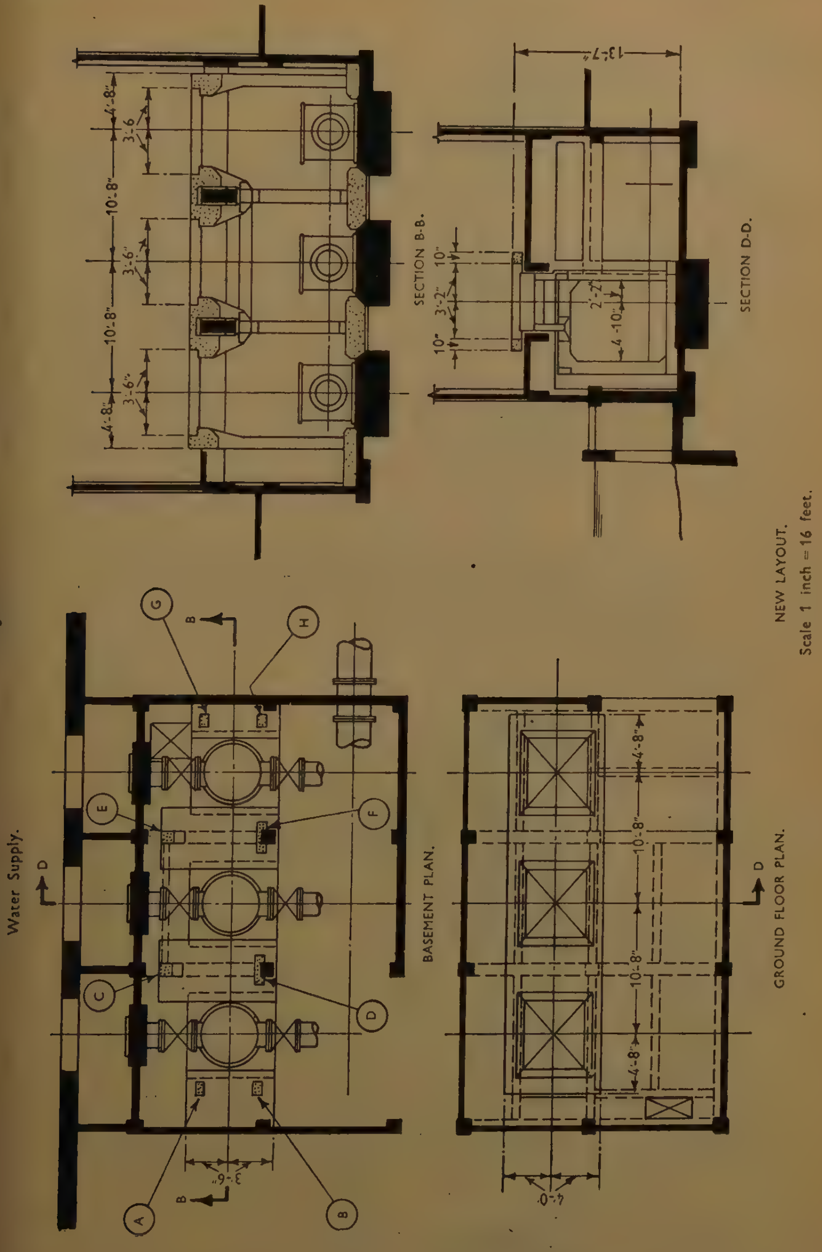
Pockets were left through the seatings so that the holding-down bolts for the bed-plates could be grouted in position as a final operation.

The penstock controls for each pump were fixed to the original floor-slab, but as only the guides were bolted to the floor it was not considered necessary to treat these in the same way as the motors. The new work




*Figs. 2.*

ORIGINAL LAYOUT.




NEW LAYOUT.
Scale 1 inch = 16 feet.

*Figs. 5 (a).*

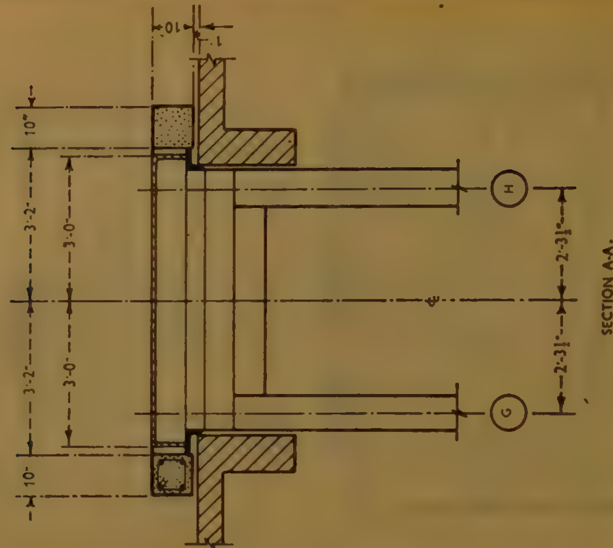
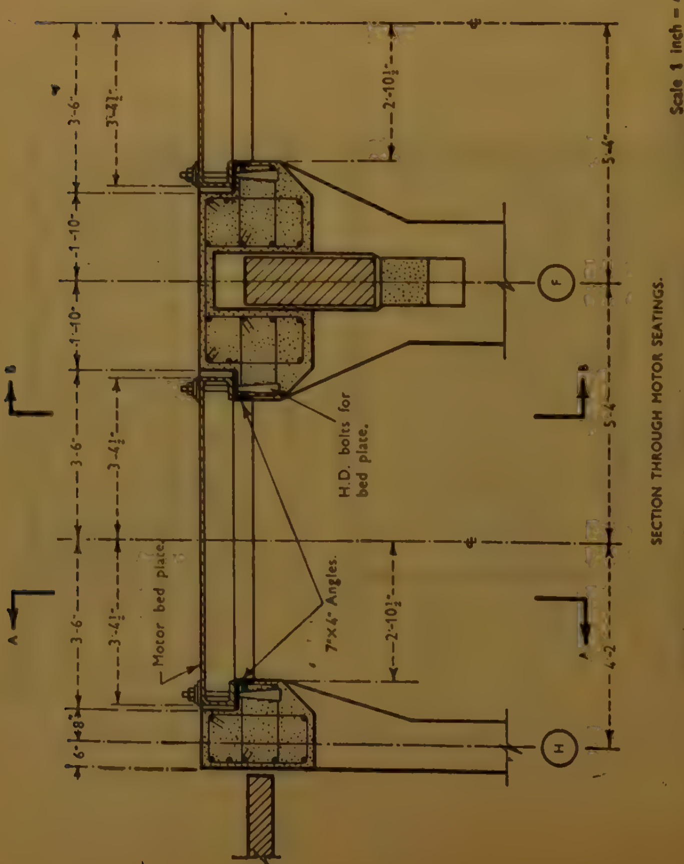
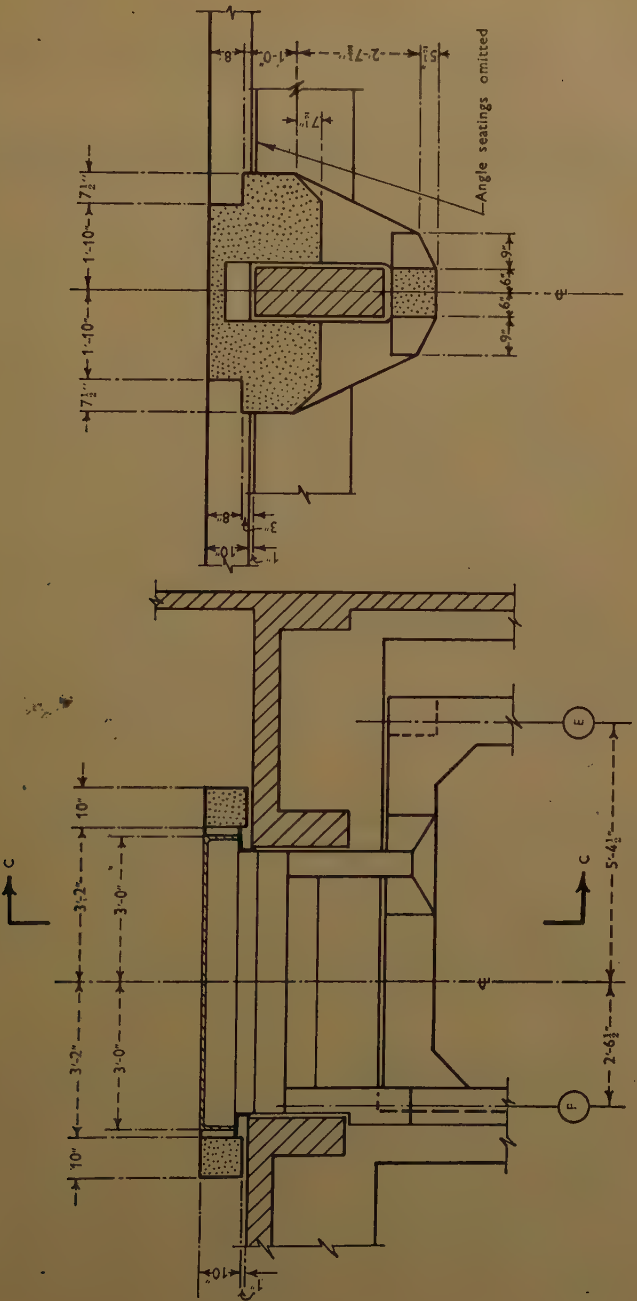



*Fig. 4.*




Scale 1 inch = 4 feet.

*Figs. 5.*




(c)

SECTION C-C.

(b)

SECTION B-B.

Scale 1 inch = 4 feet.

was isolated entirely from the existing motor-floor by means of suitable clearances, which allowed for the possibility of further settlement of the building.

The Paper is accompanied by three sheets of diagrams and two photographs, from some of which the Figures in the text have been prepared.

# OBITUARY.

GEORGE BALFOUR, J.P., M.P., was born in Portsmouth on the 30th November, 1872, and died in London on the 26th September, 1941. He was one of the pioneers of electricity-supply development in Great Britain, and through his direction of the Scottish Power Company Limited, the Midland Counties Electricity Supply Co. Ltd., the Metropolitan Electric Supply Company, Limited, the London Power Company, Limited, and the Lancashire Power Company, Limited, of all which he was Chairman, and in his capacity as a director of many other companies, he had a great influence on such development over very large areas of England and Scotland.

The firm of Balfour, Beatty & Co. Ltd., which, in conjunction with the late Mr. A. H. Beatty, he founded in 1909, and of which he was the active Chairman until his death, carried out many important engineering contracts under his leadership, including the Lochaber water-power scheme, carried out for the British Aluminium Company; the Grampian Electricity Supply Company's hydro-electric development; and the construction of the Kut barrage on the Euphrates for the Iraq Government.

In 1922, Power Securities Corporation was founded to finance electricity development and general engineering enterprises all over the world. Balfour, Beatty & Co. Ltd., became a wholly-owned subsidiary of that Corporation, of which Mr. Balfour was Chairman until his death. Mr. Balfour's name was also well known abroad, particularly in the Argentine and Uruguay; and in Malaya through his Chairmanship of the Perak River Hydro-Electric Power Co. Ltd., which he assumed at the request of H.M. Treasury.

Mr. Balfour received his early engineering training with Messrs. Urquhart Lindsay & Co. of Dundee, with whom he remained for a short time after his apprenticeship was completed. He then joined the firm of Lowden Brothers & Co. Ltd., of Dundee, of which later he became a Director, and in 1903 he went to London to join the firm of Messrs. J. G. White & Co., which was engaged largely on tramway construction. In 1909 he struck out independently and, as mentioned above, joined with Mr. Beatty, a colleague in J. G. White's, in founding the firm of Balfour, Beatty & Co. Ltd.

No mention of Mr. Balfour would be complete without reference to his political interests. Brought up in the Gladstonian Liberal tradition, his independent and inquiring mind could not reconcile its tenets with realities as he understood them, and he became a staunch Conservative. In 1910 and again in 1911 he contested the Govan Division of Lanarkshire in the Unionist interest and, though not returned, succeeded in greatly reducing the Labour majorities. In 1918 he was invited to stand for Hampstead as the Conservative and Unionist candidate; he was elected by a large majority and continued to represent Hampstead in Parliament until his death.

Mr. Balfour was elected an Associate of The Institution on the 14th April, 1931.

In 1901 he married Margaret Malloch, the elder daughter of Mr. David Mathers, J.P., of Dundee, and had one daughter and four sons, all of whom survive him.

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MAURICE DEACON was born on the 11th November, 1850, and died at Whatstandwell, near Matlock, on the 25th September, 1941. After serving his pupilage, in 1866 and 1867, with Messrs. Giles and Brookhouse, of Derby, he was engaged in making geological plans for the Coal Commission and in surveying coal and iron-ore deposits in South Wales. In 1871 he was appointed resident engineer and manager of the Plasycoed Colliery, South Wales, and in 1874 became consulting engineer to the Bath Colliery Company. In 1876 he designed and superintended the construction of a large mining plant at Bettisfield colliery, North Wales, and in 1878 started a consulting practice at Pontypool. He also acted as general manager and engineer to the Manners Colliery Company, of Ilkeston, and the Blackwell Colliery Company, and as consulting engineer to the West Hallam Coal & Iron Company. In 1896 he began his long association with the Sheepbridge Coal and Iron Co., Ltd., at first as general manager and later as managing director, which position he occupied until 1924, being largely responsible for the success of the Company. In addition he served as chairman or director of numerous other colliery and commercial undertakings and assisted in the development and management of many collieries, iron and steel works, rolling mills, foundries, coke-ovens, and by-product plants in Great Britain and abroad. He also appeared frequently as an expert witness before Parliamentary Committees in connexion with mining questions. During the 1914–18 war he acted as adviser to the War Office in connexion with the reconstruction of the French coal mines destroyed by the Germans, and was a member of a committee of French mining engineers under the French Ministry of Mines.

Mr. Deacon was elected an Associate Member of The Institution on the 7th December, 1886, and was transferred to the class of Member on the 20th December, 1892: he served on the Council from November 1914 to November 1921. He was also a Past-President of the Institution of Mining Engineers, and a Governor of Sheffield University.

He married, first, Miss Adelaide Bageley, of Cornwall, and secondly, Miss Millicent Dounes, of Shropshire, and had one son and four daughters.

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## CORRIGENDUM.

November 1941 Journal. Page 104, Footnote. *For* "Min. Proc. Mech. C.E., vol. cxci" *read* "Min. Proc. Inst. C.E., vol. cxc".

MADE AND PRINTED IN GREAT BRITAIN BY WILLIAM CLOWES AND SONS, LIMITED, LONDON AND BECCLES.

NOTE.—*Pages [1] to [12] can be omitted when the Journal is bound in volume form.*

# THE INSTITUTION OF CIVIL ENGINEERS

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*Secretary.*

E. GRAHAM CLARK, M.C., B.Sc.

# NOTICES

## No. 2, 1941—42

## DECEMBER, 1941

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## MEETINGS, SESSION 1941—42.

### ORDINARY MEETING.

Arrangements have been made for the following Papers to be discussed on the date shown below :—

1942.
Feb. 10 (Tues.) (2 p.m.). *† "**Soil Mechanics and Site Exploration**", by L. F. Cooling, M.Sc.

with

*† "**Soil Mechanics in Road and Aerodrome Construction**", by A. H. D. Markwick, M.Sc., Assoc. M. Inst. C.E.

### JAMES FORREST LECTURE.

The James Forrest Lecture will be delivered at 2 o'clock on Tuesday, 13 January, 1942, by Dr. C. S. Myers, C.B.E., M.A., F.R.S., the subject being Psychology as applied to Engineering.

### RAILWAY ENGINEERING SECTION.

1942.
Jan. 27 (Tues.) (2 p.m.). † "**Permanent Way Tests and Practice on the L.M. & S. Railway,**" by W. K. Wallace, M. Inst. C.E.

* Brief Synopses of the Papers appear at pp. [10], [11], *post*.

† Advance proofs, for those who intend to be present, will be available about a fortnight before the meetings, and copies may be obtained upon application to the Secretary.

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## GENERAL ANNOUNCEMENTS.

### THE JOURNAL.

The next Number of the Journal will be published on the 15th January.

## SUBSCRIPTIONS.

Members and Students are reminded that subscriptions for 1942 are due on the 1st January, 1942. The present subscription rates are as shown below :—

| | Class A. (London Area.) | | | Class B. (Elsewhere in British Isles.) | | | Class C. (Abroad.) | | |
|---|---|---|---|---|---|---|---|---|---|
| | £ | s. | d. | £ | s. | d. | £ | s. | d. |
| Members . . . . . . . . . | 6 | 6 | 0 | 4 | 4 | 0 | 3 | 13 | 6 |
| " (retired) . . . . . . | 3 | 13 | 6 | 2 | 12 | 6 | 2 | 12 | 6 |
| Associate Members . . . . . . | 3 | 13 | 6 | 2 | 12 | 6 | 2 | 12 | 6 |
| " " (retired) . . | 2 | 12 | 6 | 2 | 2 | 0 | 2 | 2 | 0 |
| Associates . . . . . . . | 5 | 0 | 0 | 5 | 0 | 0 | 5 | 0 | 0 |
| Students . . . . . . . . . | 2 | 0 | 0 | 1 | 10 | 0 | 1 | 10 | 0 |

Owing to the increased cost of postage and need for economy in the use of paper, members are urged to make prompt payment of their subscriptions and so save the necessity of a further application.

Attention is drawn to the fact that any contribution to the Benevolent Fund may be included in the cheque drawn in payment of The Institution subscription.

## "INGENUITY" COMPETITION.

Papers are invited from Corporate Members and Students in competition for a Prize of Twenty-five Guineas to be awarded by the Council for a description of an engineering problem and the method adopted to solve it.

The article should not exceed 2,000 words, and must describe a specific problem involving immediate action and ingenuity displayed in meeting it. The problem must have arisen in the competitor's own experience, and the action taken must have been to some extent—not necessarily wholly—his own idea. These facts must be vouched for in a satisfactory manner.

The Papers should reach the Institution by the 30th April 1942, with the MS. marked " Ingenuity " Competition in the top left-hand corner of the first page.

The Council reserve the right to publish the winning entry, or any other selected entries, and should such entries relate to engineering problems arising out of the war, The Institution would submit them to the Censor for permission to publish.

## INVITATION TO PRESENT SHORT PAPERS.

The Council are prepared to receive short original Communications of, say, 2,000 words, accompanied by two or three illustrations, for inclusion in the Journal. Such Communications should be topical in character and might deal, for example, with demolition and reconstruction problems, or

with minor constructional details, of a novel character, which would be of general interest to engineers.

## HONOURS.

The Council have much pleasure in congratulating the following members on the Distinctions conferred upon them :—

*George Medal :—*

HAMILTON, John Alexander King, B.Sc.—*Member.*

*Order of the British Empire :—*

*O.B.E.* BAKER, Professor John Fleetwood, M.A., D.Sc.—*Associate Member.*

## ROAD ABSTRACTS.

The publication of "Road Abstracts," compiled by the Department of Scientific Research and the Ministry of Transport, is being continued in 1942. By arrangement with the Institution of Municipal Engineers, members are enabled to subscribe for these Abstracts at the rate of 8*s*. 6*d*. per annum (postage free)—one-half the usual rate charged.

All subscriptions run from January.

## MR. E. GRAHAM CLARK, SECRETARY INST. C.E.

In response to suggestions, the Council have directed that a reproduction of a photograph of Mr. E. Graham Clark, Secretary of The Institution, appear as a frontispiece to this Number of the Journal (*facing p.* 107).

## TRANSFERS, ELECTIONS, AND ADMISSIONS.

Since the 23rd September, 1941, the following elections have taken place :—

| *Meeting.* | *Members.* | *Assoc. Members.* | *Associates.* |
|---|---|---|---|
| 4 November, 1941. | 3 | 98 | 2 |

and during the same period the Council have transferred four Associate Members to the class of Members, and have admitted fifty-four Students.

## DEATHS AND RESIGNATIONS.

The Council have received, with regret, intimation of the following deaths and resignations :—

DEATHS.

| | |
|---|---|
| BROOKHOUSE, Robert Harold. (E. 1883. T. 1889.) | *Member.* |
| HANSON, John Henry. (E. 1881. T. 1914.) | " |
| HUTCHINSON, William, M.C.E. (E. 1886. T. 1891.) | " |
| HOWARD, Francis Eliot. (E. 1901.) | *Associate Member.* |
| MARRINER, William Wright, B.Sc. (E. 1894.) | " " |

| | |
|---|---|
| ARMSTRONG, Francis Michael. (A. 1941.) | *Student.* |
| BYROM, Charles Cyril. (A. 1938.) | ,, |
| *HORNER, Philip Norman. (A. 1938.) | ,, |
| REUBEN, Hyam Solomon. (A. 1938.) | ,, |

* *Killed on Active Service.*

RESIGNATIONS.

| | |
|---|---|
| JACKSON, Clements Frederick Vivian, B.E. (E. 1899. T. 1920.) | *Member.* |
| ADAMS, Samuel Henry. (E. 1900.) | *Assoc. Member.* |
| BLAKEY, Othman Frank, M.E. (E. 1936.) | ,, ,, |
| FORMAN, Arthur Nelson, B.Sc. (E. 1920.) | ,, ,, |
| MAWSON, Matthew. (E. 1894.) | ,, ,, |
| PHILLIPS, Guy Addison. (E. 1933.) | ,, ,, |
| FRIEDLAENDER, Konrad Joachim. (A. 1938.) | *Student.* |
| FYALL, Douglas, B.Sc. (A. 1937.) | ,, |
| HOLME, James Edgar. (A. 1938.) | ,, |
| PEACH, Anthony John. (A. 1940.) | ,, |

## PROTECTIVE PAINTING OF STRUCTURAL STEEL.

By courtesy of the Iron and Steel Institute a number of copies of a pamphlet, "Protective Painting of Structural Steel", has been placed at the disposal of members of The Institution and may be obtained (free of charge) upon application to the Secretary, Inst. C.E. The pamphlet in question is issued by the Protective Coatings Sub-Committee of the Corrosion Committee, being a Joint Committee of the Iron and Steel Institute and the British Iron and Steel Federation.

## WAR DAMAGE ACT, 1941.

Section 7.

The War Damage Commission hereby give notice that they have, for the purpose of giving effect to directions given by the Treasury under sub-section (1) of section 7 of the War Damage Act, 1941, and under the power conferred upon them by sub-section (2) of the said section, specified the following class of works for making good war damage :—

> Works costing more than £1,000 or ten times the net annual value (if any) of the hereditament, whichever is the less, to be executed for making good the war damage sustained by any one hereditament situated within the following areas :—
>
> (*a*) The County Borough of Great Yarmouth.
> (*b*) The Boroughs of Hornsey and Lowestoft.

The War Damage Commission also give notice that they, under the power conferred upon them by the aforementioned sub-section, impose on any person, who proposes to execute for making good war damage any works (other than temporary works) of the above specified class, an obliga-

tion to inform the War Damage Commission of the proposal, and thereafter to furnish to them such particulars of the proposed works as they may require in any particular case.

In this notice a "hereditament" means either a hereditament as appearing in the rating valuation list, or, in the case of a hereditament which is exempt from rating and does not appear in that list, the hereditament so exempted, provided that where there is more than one occupation of a single building, the hereditament shall be the building.

War Damage Commission,
Devonshire House,
Mayfair Place,
Piccadilly,
London, W.1.
5th November, 1941.

## A SELECTIVE LIST OF RECENT ADDITIONS TO THE LIBRARY.

[Journals, Proceedings of Societies, etc., are not included.]

BITUMEN. LETHERSICH, W. "Mechanical Behaviour of Bitumen." 1941. B.E.A.I.R.A. 15*s*.

COAL. STUTZER, O. "Geology of Coal." 1940. Univ. of Chicago Press. 30*s*.

COSTING. NOEL-BROWN, S. J. "Costing and its Application to Government Contracts." 1941. Eng'g. Industries Assoc. 2*s*. 6*d*.

DAMS. TENNESSEE VALLEY AUTHORITY. "The Norris Project. Technical Report No. 1." 1940. Supt. of Documents, Washington. 1 dollar, 50 cents.

ELECTRICAL APPARATUS. WILSON, W. "Calculation and Design of Electrical Apparatus." 3rd ed. 1941. Chapman & Hall. 10*s*. 6*d*.

ELECTRICITY-CIRCUITS. COULTHARD, W. B. "Transients in Electric Circuits." 1941. Pitman. 25*s*.

FACTORIES AND WORKSHOPS. PULL, E. "Engineering Workshop Manual." 9th ed. 1941. Technical Press. 5*s*.

FOUNDATIONS. JACOBY, H. S., and DAVIS, R. P. "Foundations of Bridges and Buildings." 3rd ed. 1941. McGraw-Hill. 35*s*.

GEOLOGY. *See* COAL.

GEOPHYSICAL PROSPECTING. *See* OIL.

HEAT. STOEVER, H. J. "Applied Heat Transmission." 1941. McGraw-Hill. 17*s*. 6*d*.

KINEMATICS. SLOANE, A. "Engineering Kinematics." 1941. Macmillan. 17*s*. 6*d*.

LIFTS. CONTACTOR. "Electric Lifts." 1941. Newnes. 7*s*. 6*d*.

*METALS. CARPENTER, *Sir* H., and ROBERTSON, J. M. "Metals." 2 vols. 1939. Oxford Univ. Press. £5 5*s*.

—— GREENWOOD, J. N. "Glossary of Metallographic Terms." 1940. Tait Publ Co. 5*s*.

MINES AND MINING. GARDNER, E. D., and MOSIER, M. "Open-Cut Metal Mining." 1941. Supt. of Documents, Washington. 40 cents.

OIL. NETTLESTON, L. L. "Geophysical Prospecting for Oil." 1940. McGraw-Hill. 35*s*.

RAILWAYS. ANONYMOUS. "The Railways of Persia." 1941. *The Railway Gazette.* 2*s.*

RECONSTRUCTION. BOURNVILLE VILLAGE TRUST. "When We Build Again." 1941. Allen & Unwin. 8*s.* 6*d.*

—— TOWNDROW, F. E., *Ed.* "Replanning Britain." 1941. Faber. 7*s.* 6*d.*

RIVERS. STROM, H. G. "River Control in New Zealand and Victoria." 1941. Victoria State Rivers and Water Commission. No price.

SHEET METAL. COOKSON, W. "New Methods for Sheet Metal Work." 1941. Technical Press. 7*s.* 6*d.*

SHIPS. INSTITUTE OF WELDING. "Welded Ships. A Bibliography." 1941. The Institute. 3*s.*

*SMALL ARMS. HAVEN, F. T., and BELDEN, F. A. "A History of the Colt Revolver." 1940. Morrow, New York. 55*s.*

*WARFARE. MIKSCHE, F. O. "Blitzkrieg." 1941. Faber. 12*s.* 6*d.*

* The foregoing books, with the exception of those marked with an asterisk, may be borrowed from the Loan Library.

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## LOCAL ASSOCIATIONS.

The following arrangements have been made for forthcoming meetings of the Local Associations. The arrangements are in the hands of the Committees of the Associations concerned and all communications respecting them should be addressed to the respective Honorary Secretaries :—

### *Edinburgh and District Association.*

1942.
Jan. 14. "Town and Country Planning", by Mr. F. C. Mears, F.R.I.B.A.
Feb. 11. "Some Foundation Problems and Soil Action", by Mr. A. R. Pollard, B.A., M. Inst. C.E.
Mar. 11. Film illustrating "The Failure of the Tacoma Narrows Suspension Bridge."

### *Northern Ireland Association.*

1942.
Feb. 23. "Town Re-planning", by Captain Brown.
Mar. 30. Meeting to be arranged.
Apr. 27. Annual General Meeting.

### *North Western Association.*

1942.
Jan. 3. Film illustrating "The Failure of the Tacoma Narrows Suspension Bridge."
Feb. 14. "The Aesthetics of Engineering Structures", by D. T. Lloyd Jones.
Mar. 21. "Soil Mechanics applied to Site Exploration", by L. F. Cooling, M.Sc.
Apr. 25. "The Engineer's Part in Town Planning", by H. J. Manzoni, C.B.E., M. Inst. C.E.

### *South Wales and Monmouthshire Association.*

1942.
Feb. 21. Meeting to be arranged (at Swansea).
Apr. 18. Annual General Meeting (at Cardiff).

## REPORTS.

*Birmingham and District Association.*

The first meeting of the Session was held on Thursday, 9 October, when the Chairman, Mr. B. C. Hammond, M. Inst. C.E., gave an informal address on "Unusual Problems arising in Engineering Maintenance."

*Edinburgh and District Association.*

The following meetings have been held: Wednesday, 15 October, when the Chairman, Mr. W. A. Macartney, M. Inst. C.E., delivered his Inaugural Address; Wednesday, 12 November, when a Paper on "The Callender-Hamilton Unit-Construction Bridge" was read by Mr. C. O. Boyse, B.Sc., M. Inst. C.E.

*Northern Ireland Association.*

The opening meeting of the Session was held on Monday, 27 October, when the Chairman, Mr. R. D. Duncan, B.Sc., M. Inst. C.E., delivered his Inaugural Address.

*North Western Association.*

On Saturday, 8 November, Mr. Robert Slater, M.Sc., Assoc. M. Inst. C.E., read a Paper on "Road Experiments in the Design of Thin Bituminous Surfacings."

*South Wales and Monmouthshire Association.*

The first meeting of the Session was held at Cardiff on Saturday, 8 November, when a film illustrating "The Failure of the Tacoma Narrows Suspension Bridge" was exhibited.

*Southern Association.*

On Thursday, 9 October, a Paper on "Problems in the Construction of Air Raid Shelters" was read by Mr. Frederick Webster, M.C., M.Eng., M. Inst. C.E.

*Yorkshire Association.*

The Chairman, Mr. W. Gordon Lees, B.Sc., M. Inst. C.E., delivered his Inaugural Address at a meeting at Leeds on Saturday, 8 November.

*Synopsis of*

## CHAIRMAN'S ADDRESS.

EDINBURGH AND DISTRICT ASSOCIATION.
*Meeting,* 15 *October*, 1941.

### "The Civil Engineer in the New Era."

By WILLIAM ALLAN MACARTNEY, M. Inst. C.E.

There are bound to be changes in the municipal, social, industrial, and national spheres after this war, and the Civil Engineer will have a large part to play. The speaker suggested and dealt in detail with certain spheres which, in his opinion, would demand the best that we could give.

*Town Planning.*—The proposals of the Government up to date were analysed and their failure to meet the general demand of a "Central Planning Authority" was pointed out. The work of the War Damage Commission and its powers to stop plans for rebuilding until Local Authorities in "blitzed" areas were consulted, was emphasized.

The work of a "Central Planning Authority" would be to survey the national outlook in regard to agriculture, industry, transport, and many other things, in addition to regional matters such as water-supplies, sewage-disposal, etc. Until this was done, there could be no, what might be called, local town planning. Meantime the Engineers to all Local Authorities should be collecting the necessary information and data to allow them to proceed with their Town-Planning Schemes as soon as the national reports were completed.

*Rehabilitation of "Blitzed" Areas.*—The areas referred to numbered at least a couple of dozen, and their restoration would raise many problems. It was possible that many industries would desire to transfer elsewhere in order to secure an easier supply of raw materials, cheaper transport, or better housing facilities for their employees. The designing of new factories and all their complements was work which the civil engineer could do well, and the desirability and possibility of making these more aesthetically pleasing was pointed out, reference being made to the very able Paper by Dr. Faber [1] on that subject. In that connexion, the work of Thomas Telford, who was both architect and engineer, and the beauty of his bridges and other works, were remarked on.

*Traffic Congestion.*—The number of vehicles licensed increased from 1,729,505 in 1926 to 3,093,884 in 1938, whilst the length of roads, which was now 179,630 miles, had increased by only a fraction of 1 per cent. The causes of congestion were examined, and the great loss which it caused to business interests was pointed out. No one solution was, however, possible,

[1] Oscar Faber, "Aesthetics of Engineering Structures." Journal Inst. C.E., vol. 16 (1940–41), p. 139 (April 1941).

but the country could do with a few fast motor roads and ought to have many new trunk roads, numerous by-passes, bridges widened and strengthened, and hundreds of miles of main roads widened, provided with footpaths and drains, regraded and superelevated and controlled by proper junctions.

*River Pollution and Sewage-Disposal.*—The powers of Local Authorities and the responsibility of manufacturers were discussed, and suggestions were made for dealing with the problem of river pollution. The problem of sewage-disposal was still the sludge problem, which in these days was one of the greatest importance to the nation.

*Rural Water-Supplies.*—The grants made for this object were terribly inadequate and until the State could make substantial grants available, this crying problem would remain unsolved in large areas of the country where the density of population was low.

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## ABSTRACTS OF PAPERS FOR DISCUSSION.

The following Papers will be brought forward for discussion on the date indicated in the margin of the abstracts, and will be published, with reports of the oral and written discussions upon them, in the Journal. Members desiring to take part in the consideration of the Papers should apply forthwith for advance copies, which will be forwarded as soon as they are ready. Applications for proofs should be made on postcards, quoting the numbers of the Papers.

A period of about 3 months from the date of publication of the Papers in the Journal is generally allowed for written communications, which should be :—

(*a*) As concise as possible, entirely relevant to the subject-matter of the Paper, and consist of not more than 1,000 words ;

(*b*) Written legibly or typed with the lines openly spaced.

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**Date of Discussion 10/2/42.**

*Paper No. 5292.*

### "Soil Mechanics and Site Exploration."

By Leonard Frank Cooling, M.Sc.

*Abstract.*

The Paper outlines the theoretical and practical considerations that soil mechanics indicates should be taken into account in the examination of sites for engineering structures. Such methods are of particular use when the engineer meets soil conditions radically different from normal, o

where the proposed structure presents novel features. Direct and indirect methods of soil exploration are described and the theoretical considerations furnished by soil mechanics as well as practical considerations are given, with illustrations.

The application of the technique to common stability problems is considered with reference to four specific examples that have been studied at the Building Research Station.

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*Paper No. 5297.*

Date of Discussion 10/2/42

## "Soil Mechanics in Road and Aerodrome Construction."

By Alfred Herbert Dorlencourt Markwick, M.Sc., Assoc. M. Inst. C.E.

*Abstract.*

The Paper describes important differences between road and aerodrome problems arising from the differing nature of the wheel-loads. Foundations, earthworks, soil stabilization, and retaining walls are discussed. The site is first surveyed and classified by means of tests and soil mechanics, and then loading conditions and the bearing capacity of the soil and of paved runways are ascertained. The maximum wheel loads of aeroplanes are much higher than those of road vehicles. The method of ascertaining the bearing capacity of the soil in terms of its mechanical properties, and the application of dimensional analysis to the bearing capacity of the soil under aeroplane tires, are described in two appendixes. It is shown that the pressure in and beneath the runways can be calculated by the method used for concrete roads, with slight modifications. The best methods of aerodrome drainage and embankment construction are discussed.

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